PERFORMANCE AND COST EFFECTIVENESS OF CENTRALIZED INFILTRATION BASINS AND DECENTRALIZED LOW IMPACT DEVELOPMENT PRACTICES IN A SEMI-ARID URBAN WATERSHED

by

Marianna Dorota Sochanska

A thesis submitted to the faculty of
The University of Utah
in partial fulfillment of the requirements for the degree of

Master of Science

Department of Civil and Environmental Engineering

The University of Utah

August 2014

UNIVERSITY OF UTAH GRADUATE SCHOOL

STATEMENT OF THESIS APPROVAL

The thesis of Marianna Dorota Sochanska

Has been approved by the following supervisory committee members:

Steven Burian, Chair — 05/06/2014 Date Approved

Christine Pomeroy, Member — 05/06/2014 Date Approved

Brian James McPherson, Member — 05/06/2014 Date Approved

and by Michael Barber, Chair of

the Department of Civil and Environmental Engineering.

and by David B. Kieda, Dean of The Graduate School.

ABSTRACT

Low impact development (LID) is gaining popularity for its ability to revert developed landscapes to their historical hydrology and thus enhance sustainability. Utilizing a case study of a low impact development in the Salt Lake City Valley in Utah, a comparison of the effects of centralized infiltration to different LID technologies was conducted. Stormwater simulated rainfall-runoff models were created using the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM). To evaluate several scenarios models with different features were developed: no controls (developed land with no stormwater control); centralized infiltration (large infiltration basins); rainwater harvesting (rain barrels); porous pavement; bioretention; and a comprehensive model with all of the LID features together.

The results of the models show statistically significant ($p<0.05$ and t stat>t critical) decreases in average annual total flow volume, average annual mean flows, and average annual peak flows from the no controls model. For all reductions, a comparison of model performance in wet and dry years (a classification based on precipitation amounts) was completed. This revealed that the centralized infiltration model performs better in dry years, the comprehensive LID, porous pavement, and bioretention models perform better in wet years, and finally that rainwater harvesting generates similar reductions in outflow regardless of the type of year. A cost analysis of the models was conducted in order to help quantify the use of the technologies. In terms of costs per

volume reductions, bioretention proved to be the least expensive option, followed by rainwater harvesting, centralized infiltration basins, comprehensive LID, and finally porous pavement.

Based on these results, the final recommendation is that projects should first consider bioretention and then rainwater harvesting as options for stormwater management. Porous pavement is an effective choice, however, its cost is a deterrent. Comprehensive LID model provides successful reductions but at a higher cost due to the inclusion of porous pavement. Centralized infiltration is still a good choice for new developments since it is shown to be very effective, however the land amount it requires for implementation may be a deterrent for using this type of stormwater management in retrofitting projects.

I dedicate this to my family.

TABLE OF CONTENTS

ABSTRACT .... **iii**

LIST OF FIGURES .... **viii**

ACKNOWLEDGEMENTS .... **x**

INTRODUCTION **1**

Research Questions 3

LITERATURE REVIEW **4**

Sustainability 4
LID Technology 5
LID Costs 6

METHODS **8**

Case Study Description 9
Utah Climate 10
SWMM 11
Description 11
Precipitation Data 12
GIS Data 13
Verification 16
SWMM Models 16
Statistical Analysis 18
Cost Estimates 18

RESULTS **20**

Long-Term SWMM Model Results 20
Overall Results 20
Cumulative Volume 21
Average Total Annual Volume 23
Average Total Annual Volumes by Type of Year 23
Exceedance Probability of Annual Volumes 23
Average Annual Mean Flows 26
Average Annual Mean Flows by Type of Year 27

Exceedance Probability of Annual Mean Flows 29
Average Annual Peak Flows 30
Average Annual Peak Flows by Type of Year 32
Exceedance Probability of Annual Peak Flows 32
Long-Term Results Summary 34
Design Storm SWMM Model Results 35
Cost Estimates 38

DISCUSSION 42

SWMM Model Results 42
Internal/External Threats to Validity 44

CONCLUSION 46

Analysis Results 46
Future Research 47

APPENDICES

A PRECIPITATION ANALYSIS 48

B GIS FILES 52

C SWMM MODEL PARAMETERS 54

D SWMM MODELS 60

E VERIFICATION 67

F STATISTICAL SIGNIFICANCE 80

LITERATURE CITED 135

LIST OF FIGURES

1. Daybreak, Utah Subwatershed Delineation 14

2. Daybreak, Utah Subwatersheds with Infiltration Basins 15

3. Cumulative Volume of Outflows (ML) for the Long-Term Data Set 22

4. Annual Average Total Volumes of Outflow (ML) 24

5. Annual Volumes of Outflow by Type of Year 25

6. Exceedance Probability of Average Annual Runoff Volume 26

7. Average Annual Mean Flows 27

8. Average of the Annual Mean Flows by Type of Year 28

9. Exceedance Probability for Average Annual Mean Outflows 30

10. Annual Average Peak Flows 31

11. Average of the Annual Average Peak Flows by Type of Year 33

12. Exceedance Probability of Average Annual Peak Flows 34

13. 2-year 24-hour Design Storm Hydrograph 37

14. 10-year 24-hour Design Storm Hydrograph 37

15. 25-year 24-hour Design Storm Hydrograph 38

16. Exceedance Plot of Total Event Precipitation 50

17. 100-year 24-hour Design Storm 69

18. Outflow Results for CI Model from 100-year 24-hour Design Storm 69

19. 2-, 10-, and 25-year 24-hour Design Storms 70

20. Rational Method Results for FoundersParkVillage_South Subwatershed 72

21. Rational Method Results for EastlakeVillage_6 Subwatershed 73

22. Rational Method Results for DaybreakPkwy Subwatershed 74

23. CN Method Results for FoundersParkVillage_South Subwatershed 77

24. CN Method Results for EastlakeVillage_6 Subwatershed 78

25. CN Method Results for DaybreakPkwy Subwatershed 79

ACKNOWLEDGEMENTS

This thesis and research would not have been possible without the help of my professors, family, friends, and fellow graduate students. I would not be where I am today without my advisor, Dr. Steve Burian. His academic support is unparalleled and I will always be extremely grateful. I would also like to thank my fellow graduate students and my friends for their continued encouragement, assistance, and understanding. Most of all I want to thank my amazing family, your love and unwavering support of the twists and turns of my life has been nothing short of incredible. Thank you for giving me this phenomenal gift of my education.

INTRODUCTION

Finding the best approach to stormwater management is a multilayer issue. Management tactics differ based on individual locations and the climate of a given region. In addition, cost is always a factor that must be considered. Currently the trend in stormwater management is shifting towards a more holistic approach. Especially in the western United States, new developers are seeking more water responsible development designs and older communities are funding retrofitting projects to create communities that are more sustainably conscious. The question remains as to what stormwater management strategies work best in the semi-arid western U.S.

One of the larger effects of urbanization is seen with the increase of impervious surface areas. Increasing impervious area decreases the amount of infiltration that is possible, which increases stormwater runoff in terms of total volume and peak discharges (Leopold 1968). Spreading urbanization has modified natural hydrology and it impacts historical flow paths of water, the patterns of infiltration, and the native soils and their characteristics (Poff 1997). Traditional stormwater management collects and conveys stormwater runoff offsite through storm drains, pipes, or other conveyances. Stormwater management in the State of Utah has historically focused on providing adequate flood control measures. Thus stormwater runoff generally flows directly into streams and rivers without any treatment. In urbanized areas, such as Salt Lake City, this stormwater runoff carries various organic and inorganic pollutants and these nonpoint sources of pollution negatively affect the streams, lakes, rivers, and reservoirs in the state (UDEQ 2013).

Stormwater management practices are beginning to shift away from centralized approaches, such as regional detention strategies that focus on flood control (Young 2011), due to the opportunities for increased performance and enhanced sustainability that decentralization can provide (Burns 2012, Jia 2012). Smaller decentralized methods are now being tested and applied to better mimic the natural hydrology of a given area by seeking to increase infiltration or capture and reuse of stormwater (Jia 2012). These stormwater management practices are called low impact development (LID) practices. The overall goal of LID is to decrease stormwater runoff, increase infiltration, promote evapotranspiration, prevent erosion, and treat and decrease pollutant loads (Bedan 2009, Freni 2010).

This new focus aims to create hydrologically functional landscapes that are able to better function without large-scale infrastructure (Prince George's County 1999). The decentralized practices attempt to maintain the predevelopment levels of ecological and geomorphic features of receiving waters, while still maintaining the ability to remove or reduce increased stormwater runoff volume and pollutants that are generated in urban environments (Burns 2012, Freni 2010, Palhegyi 2010). The United States Environmental Protection Agency (EPA) has begun a campaign to encourage the use of LIDs in arid and semi-arid climates due to their ability to target several stormwater management aims. (EPA 2010). Regulation changes that will reduce stormwater discharges and increase stormwater quality are now being favored. Overall, the focus is shifting to restore the natural water balance.

This thesis seeks to compare centralized infiltration basins with distributed LID features as stormwater management techniques for a semi-arid climate. Although the transition from traditional to site-scale management is currently underway in practice, the long-term implications of this change remain relatively unknown. Additionally, the potential to enhance the sustainability of regional systems throughout the entire lifetime must be studied with respect to the climatic and urban structure. As such, a cost analysis was included to compare the centralized and decentralized management options to show the tradeoff between the costs of implementation and the hydrologic benefits of each scenario. This work contributes to both current and future decision making with respect to both local and regional sustainable stormwater management.

To generate this comparison, the case study of a new sustainably designed development of Daybreak, South Jordan, Utah was utilized. Stormwater models were created with the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM) to evaluate the impact of distributed, on-site infiltration practices and technologies on the rainfall-runoff response for the community. The hypothetical models were created in a way that the LID implementation would be both spatially and economically feasible. The following research questions were addressed with this thesis.

Research Questions

1. Determine whether replacement of the large infiltration basins with LID are capable of attaining the comparable stormwater management benefits, such as volumetric, mean flow, and peak flow reductions over the long-term record.
2. Determine which scenario is more cost-effective.

LITERATURE REVIEW

Sustainability

Water resource management is becoming a serious issue of the present time. Populations are increasing, urban areas are increasing in population density, and the climate appears to be shifting (Bierwagen 2010, Utah 2001). Being able to ensure adequate water supply, stormwater management, and flood mitigation for the public, while still protecting natural resources, is creating a change in the way cities manage their water resources and design/build new additions to their communities (Donofrio 2009). New urban developments are beginning to protect their natural hydrological features in order to mitigate the adverse effects of urbanization on receiving waters (Berke 2003). Managing water is a growing field and it is moving towards sustainability.

Sustainability can be defined based on the social, economic, and environmental aspects of any given community (Mapes 2011). The general idea of sustainable communities is to create an integrated neighborhood, working space, and areas for exercise and social interaction. While it may not be precisely defined, sustainable communities tend to be more environmentally friendly due simply to the fact that this is what consumers are currently demanding. Many cities, new developments, and retrofitting projects are adapting new green methods and implementing LID technology (Mapes 2011, Donofrio 2009, Flakne 2012, Berke 2003).

LID Technology

In the past the typical approach to urban design focused on flood mitigation and water supply. Infrastructure systems were put in place to quickly convey stormwater away from urban areas to prevent flooding. This type of traditional stormwater management worked to decrease the increased peak flow rates from urban development through the use of large stormwater detention basins or ponds (EPA 2000). These detention areas hold excess water and then release it so that the peak outflow rate does not exceed the historical flow rate for the area and storm.

The current fluctuation in water resources management is towards a better balance between urbanized areas and the natural hydrological systems (Prince George's County, 1999, USEPA, 2000, 2007). Stormwater treatment and management are now being integrated into the landscape, instead of being speedily moved away off site (Donofrio 2009). This type of control at the source minimizes the overall amount of stormwater and the pollution that is associated with conventional stormwater conveyance systems (Berke 2003). By using smaller, decentralized methods of managing stormwater, such as LID, this creates an approach that is more capable of mimicking hydrologic processes of the land before it was developed. Retention, detention, infiltration, and treatment of stormwater runoff are capable of being conducted at the source.

The present focus has shifted to smaller scale decentralized strategies to manage stormwater runoff in order to attenuate peak runoff and better handle, and decrease, runoff volumes. These types of strategies better mimic natural processes of what happens to stormwater. LID is a specific type of land development that seeks to minimize the impacts of urbanization on stormwater runoff and to more closely mimic predevelopment conditions (slower flow that is more spread out, more capable of being stored in the ground, and thus more capable of infiltration).

LID Technology has been proven to have positive effects on protecting watersheds and helping them to maintain their natural water balance (Berke 2003, Bedan 2009, Williams 2006, Holman-Dodds 2003). The use of LID features shows a significant decrease in storm flow depth and a decrease in certain pollutants, as compared to the traditional watershed (Bedan 2009). Other studies have shown lower runoff volumes from low impact urbanization as a result of the ability of LID to infiltrate stormwater (Holman-Dodds 2003, Brander 2004, Damodaram 2010, Hood 2007, Jia 2012, Montalto 2007); these do work best for small storm events. Creating a community with a combination of LID and large detention basins can account for smaller, more frequent storms and the larger, more intense storms (Holman-Dodds 2003, Damodaram 2010). Although LID features may not lower peak flow to the same extent as large detention basins for design storms, it does mimic the predevelopment timing of flows (Damodaram 2010).

LID Costs

Research by the USEPA has shown that LID technologies tend to be more expensive to install than conventional stormwater infrastructure, however, it is actually more cost effective on a volumetric basis since it is capable of storing more water on site (Heaney 2002). Cost analysis studies have shown that LID systems have lower maintenance costs and required personnel work hours, along with higher water treatment capabilities than conventional systems (Houle 2013). While the cost-effectiveness of implementation of LIDs may vary depending on each individual situation and resulting volumetric flow reductions, it is possible that LIDs can be a cost-effective alternative if properly designed and implemented (Montalto 2007). Due to this, it is hard to definitively determine which of the LID Technologies is the most cost-effective. Each situation must be approached individually to determine the best solution for a given community and climate in terms of desired costs and reductions.

METHODS

In order to create a comparison between centralized and decentralized methods of stormwater management, a case study was created for the low impact development of Daybreak, Utah. One of the goals of this development was to retain all of the precipitation that it receives on-site, up to the 100-year storm, and allow it to infiltrate into the ground instead of being directed off site. This was accomplished by creating Daybreak as a cluster development, with houses built closely together with large common areas that double as infiltration basins. This is a unique community that was designed and built from the ground up. This type of development seeks to minimize nonpoint sources of pollution, reduce impervious surface area, promote more natural drainage that in turn decreases stormwater flow and soil erosion, optimize stormwater management, create areas for recreation, stimulate/encourage social interaction, encourage walking, decrease travel times for services, and encourage the growing of community gardens. In addition, the parks and open spaces have a second role as water detention areas that can intercept runoff, encourage infiltration, and trap nutrients.

Due to its design, it was possible to look at the stormwater generated in this community and then determine the amount and type of LID features that could possibly replace its large infiltration basins. A stormwater runoff model for Daybreak was built using two computer systems, Geographic Information System (GIS) and Storm Water Management Model (SWMM). One predevelopment and several post-development scenarios were modeled in order to be able to compare and contrast the effects of urbanization and LID implementation.

Watershed delineations for Daybreak were created in GIS. In addition, the GIS model was utilized to characterize the area of the watershed and its characteristics (such as land cover, land use, and soil types). From this, SWMM models were created in order to determine the response of the Daybreak watershed to precipitation events. The SWMM models were constructed using a variety of data resources, tools, and analysis methods. The following sections will discuss the models used for all simulations, the data used for model input and watershed characterization, and the respective data sources.

Case Study Description

Daybreak is a planned community built by Kennecott Land (a subsidiary of Kennecott Utah Copper, Rio Tinto) in South Jordan, Utah. It was built with the goal of being sustainable, environmentally friendly (each home is Energy Star certified), to have zero stormwater runoff, and for each home to be within a 5 minute walk of a major amenity.

Kennecott previously used this land for the South Jordan Evaporation Ponds (25 ponds that covered a combined 530 acres) for mining and flood management purposes until ceasing this operation in 1986. This land was heavily impacted and required environmental remediation, restoration, and reclamation in the mid-2000s before development. This allowed for the developers to be creative in their design since they did not need to account for current features and topography.

The information on Daybreak was gathered through marketing material available on the community's website (http://www.daybreakutah.com). While the community has many features, which allow it to be called sustainable, the main focus of this paper will be on the stormwater management of Daybreak. A unique feature of this particular community is that Daybreak was designed to retain 100% of stormwater that falls on the site (for up to a 100-year storm) with no connections to the municipal storm sewer system (confirmed with SWMM design storms – see Appendix E). All of the precipitation it receives is stored on-site in infiltration basins strategically placed throughout the community in the extensive parks and open spaces. The stormwater that these infiltration basins receive is then infiltrated into the ground.

The integration between stormwater management, natural systems, and recreational areas creates a distinctive mix of services and amenities. According to Daybreak, their engineers estimate that the community will save over $70 million due to the elimination of municipal impact fees and the dramatic reduction in conventional conveyance infrastructure.

Utah Climate

Water resource management is largely dependent upon the climate of a region. According to the Köppen-Geiger climate classification, Utah has a semi-arid climate. This means that this region typically experiences hot, dry summers and cold winters with an average annual precipitation between 12.7 to 38.1 centimeters.

Stormwater management strategies will work differently in different climates due to the variable hydrologic responses of watersheds. Since the use of LIDs in Utah has not been widely studied, this is an important field for the expansion of research to show the benefits of changing stormwater management techniques.

SWMM

Description

A detailed hydrologic analysis was completed using the SWMM Version 5.0. This model was specifically chosen for its ability to assess the impacts of urban development on the hydrology of a watershed, and its ability to model different stormwater management strategies with which these impacts could be mitigated. SWMM separates the modeling into four components: a basin model, conveyance routing, meteorological model, and control specifications. The program is applicable to this specific urban modeling effort due to its ability to model urban storm drainage infrastructure in combination with overland flow and low impact development (LID) controls. SWMM is also capable of simulating single event or continuous simulation of runoff volumes and peak flows.

All model simulations were run using Green-Ampt (infiltration method) and Kinematic Wave (routing method). The continuous model runs were from 9/01/1951 to 9/01/2011. The reporting time step for all models is 1 hour; runoff time step is 5 minutes for dry weather and 5 minutes for wet weather; and the routing time step is 30 seconds.

Due to the large amount of land remediation and manipulation of this particular area from its previous use, it was treated as if the predevelopment land is identical to the present configuration. So all models are run based upon the geographical nature of the land as it is now, and not how it originally may have been configured.

The developed site drains to a number of infiltration basins via overland flow and stormwater pipes. The use of dry wells in several of the basins was not studied in this paper. The total stormwater outflow of the watershed as a whole was compared to the predevelopment scenario in order to see the effects of urbanization and the use of LIDs. A comparison will be made between the developed site with and without the infiltration basins, and also with LID controls.

Precipitation Data

Precipitation is the primary source of water input into the hydrologic cycle. Here it is assumed that all precipitation is rain, and snow is not accounted for in this model due to the typically low snowfall in this area. The precipitation data were tabulated in Excel and analyzed. The yearly precipitation data were then categorized by type of precipitation that fell in each year (“wet”, “average”, or “dry”). These results were confirmed with the Palmer Drought Severity Index (Appendix A).

Sixty years of continuous precipitation was obtained from the National Oceanic and Atmospheric Administration’s (NOAA) National Climatic Data Center’s (NCDC) online database. The precipitation record is from the Salt Lake International Airport, which is approximately 16 miles north and 550 feet lower in elevation than the Daybreak watershed. The SWMM model was run continuously for the data available from 1951-2011. The data obtained from the Salt Lake City International Airport gage have the longest range and the best maintained data available for the area. Therefore they were chosen for use in the SWMM model.

A continuous data set for the precipitation data was specifically chosen in order to be able to model long-term behavior of the models. This historical time-series allows the model to show the watershed’s hydrological response through a variety of rainfall events in addition to the dry periods in the record. This allows for a broader potential application than analysis of several chosen design storm events.

In addition to the long-term precipitation data, several design storms were chosen for verification purposes. The design storms are SCS Type II, 24-hour synthetic rainfall distributions and the rainfall depth for each storm was determined with NOAA's Atlas 14 online database. The design storm depths are shown below in Table 1.

GIS Data

Esri's GIS Version 10 was used to display and compute several SWMM model input parameters. A number of GIS data layers (DEM, LiDAR, and aerial data) were obtained from the Automated Geographic Reference Center (AGRC) operated by the Utah State Government (www.gis.utah.gov). The data obtained through the AGRC website included soil type distribution, local contours, and aerial photography of the site (Appendix B).

The overall Daybreak area was delineated in GIS based upon the information provided on the Daybreak/Kennecott website. The subwatershed delineations were done based upon a site visit, the placement of stormwater drains, infiltration areas, and the geographical elevations and slopes. The final GIS delineation for the current development along with subwatersheds is shown in Figure 1. The large infiltration basins in the development are shown in Figure 2. A complete list of parameters and hydrological characterization, as determined for SWMM model through GIS, is in Appendix C.

Table 1. NOAA Atlas 14 Design Storm Depths

Return Period (years)	Depth (mm)
2	34
10	47
25	59
100	61

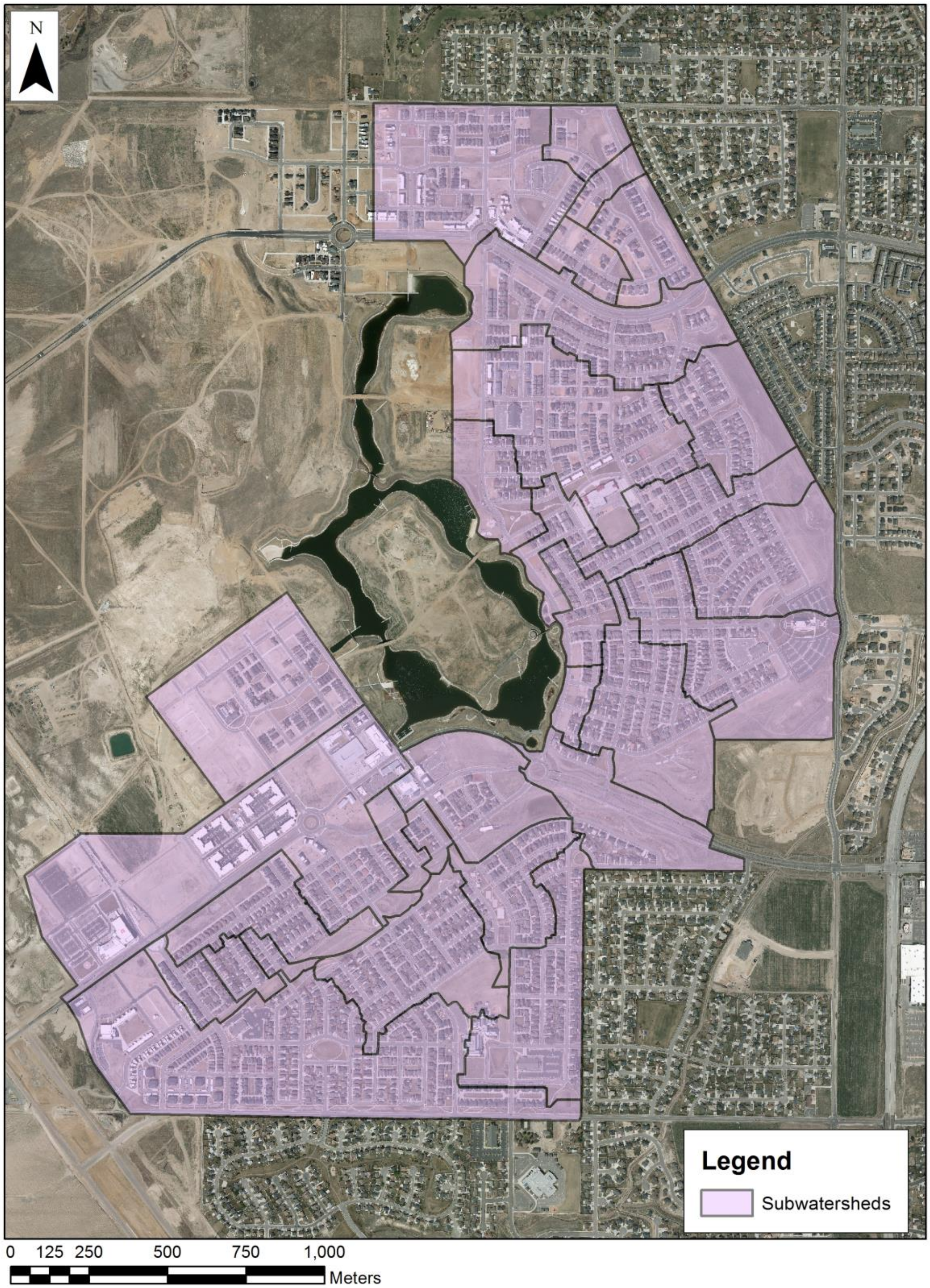

Figure 1. Daybreak, Utah Subwatershed Delineation

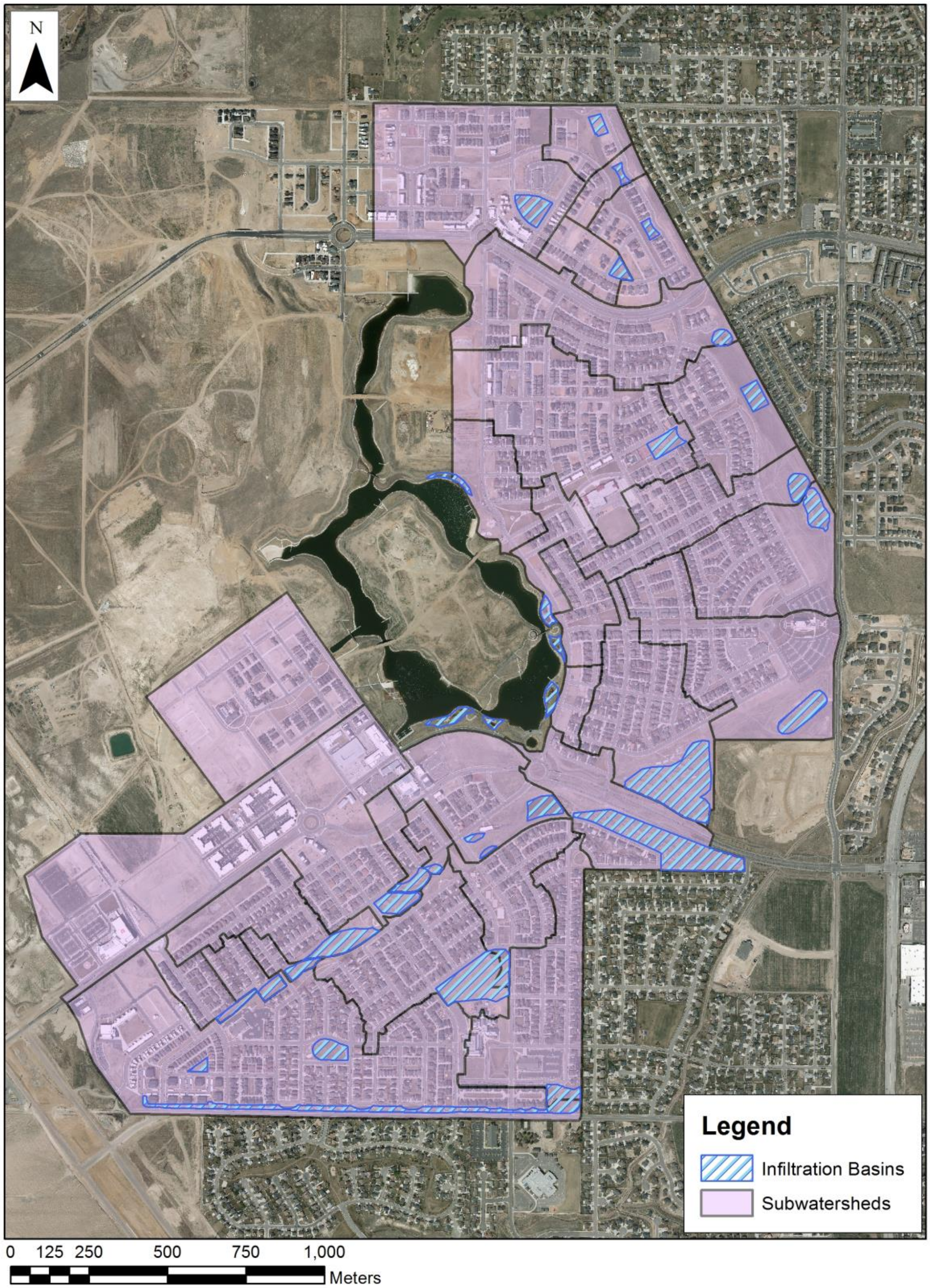

Figure 2. Daybreak, Utah Subwatersheds with Infiltration Basins

Verification

The outflow data are calculated by SWMM and are based upon the precipitation along with the parameters chosen in each individual subwatershed. Due to the lack of runoff data for this area, the Rational Method and the Curve Number Methods were utilized to verify the peak runoff values obtained. Based on the results of these two methods (shown in Appendix E), the SWMM models could be confirmed.

SWMM Models

The first model to be built was the predevelopment model. This model shows the natural characteristics and hydrological responses of the land with no urbanization. Several post-development models were built to then show the effects of urbanization, and the various methods with which it is possible to mitigate these effects. All of the SWMM models are listed in Table 2.

The predevelopment watershed was created based upon the current boundaries of Daybreak, Utah. This watershed contains a total of 413.2 hectares. After conducting a site visit, this watershed was further subdivided into subwatersheds to accurately reflect the development and its current hydrological state. All of the soil, slope, and areas for each of the subwatersheds is the same in every model, these values are listed in Appendix C.

The No Controls (NC) model shows the currently developed land without any stormwater infrastructure or LID features. This model shows the effect of urbanization without any controls on it. The second post-development model was created to most closely reflect this community as it presently exists, this is the Centralized Infiltration (CI) model.

Table 2. Modeled Scenarios

SWMM Model	Short Name	Feature
Predevelopment	P	Pre-existing land and characteristics
No Controls	NC	Developed land with no stormwater management
Centralized Infiltration	CI	Large infiltration basins
Rainwater Harvesting	RH	No Controls base model but with the addition of rain barrels
Porous Pavement	PP	No Controls base model but with the addition of porous pavement
Bioretention	BR	No Controls base model but with the addition of bioretention gardens
Comprehensive LID	CLID	No Controls base model but with the addition of rain barrels, porous pavement, and bioretention gardens

In order to test the research questions, various LID features were all added to the No Controls SWMM model. Stormwater infrastructure was not added to these models in order to show the effects of removing the big infiltration basins and attempting to replace them with only LID technologies. After testing many LID scenarios and models, the four best examples were chosen. These four utilized LID models include the following features: the Rainwater Harvesting (RH) model contains 190 liter rain barrels at every home; the Porous Pavement (PP) model replaces all driveways and sidewalks in the community with porous pavement; the Bioretention (BR) model replaces parking strips and portions of parks with bioretention gardens; and the Comprehensive LID (CLID) model incorporates all of the previously mentioned LID features together into one model. All models are explained in further detail in Appendix D.

Statistical Analysis

Statistical analysis was conducted using the student t-test to compare the outflow characteristics produced from the models. First an f-test of variance was done to determine equal or unequal variance. Based on the results of the f-test, the appropriate t-test was identified. The chosen t-test was then used to determine the significance between the No Controls model and each of the post-development models for the total annual volumes of outflow. Significance between the models for annual mean flows and annual peak flows was also investigated. Finally the statistical significance between the effects of the models (annual mean flow, annual peak flow, and annual total volumes) in wet versus dry years was examined.

The t-test searches for differences in mean values of a data set. The null hypothesis for all of the t-tests was “0,” i.e., the difference between the mean values for sample sets is zero. The alpha value was chosen to be 0.05, indicating a 95% confidence interval. Two metrics were used to determine the statistical significance: 1) ρ two-tail $<$ 0.05 and 2) t Stat $>$ t Critical two-tail. Appendix F contains all of the statistical analyses done in this thesis.

Cost Estimates

The costs for porous pavement and bioretention gardens were created with the WERF BMP and LID Whole Life Cost Models: Version 2.0 (WERF 2009). This is a spreadsheet that can be used to identify capital, operation and maintenance, and whole life costs for certain LID techniques. The costs for the 190 liter rain barrels were calculated for the community based on the total number of houses in the model. The average market value was used for the cost of the barrels and parts, and 1 hour of labor to install was included as well. A spreadsheet for rain barrel costs (Low Impact Development Center 2007) was modified in order to obtain a breakdown of necessary parts and costs for the rain barrels. The cost for the large infiltration basins was estimated after consultation with the Engineering Firm who helped design them (Ryan Cathey, NV5. Personal Communication. March 2014). The actual cost could not be obtained or confirmed from Kennecott.

RESULTS

Long-Term SWMM Model Results

Overall Results

Due to the lack of any stormwater management infrastructure in the NC model, it produced the largest total volume of outflow (41361 ML). Each of the models showed a reduction of total volume of outflow compared to the NC model. Percentages of reduction were calculated for each model based on comparing model performance to that of the NC model. This percent reduction was determined in order to best show the comparison between the NC model and the models with infiltration and LIDs.

Three categories were compared for each of the models: average annual total volumes of outflow, average annual mean flows, and average annual peak flows. The overall resulting percent reductions from the NC model are shown in Table 3. As a comparison to these values, this site at its predevelopment conditions has annual average volumes reduced by 99%, annual average mean flows that are 93% lower, and the annual average peak flows decreased by 98% (when calculated as a reduction from the NC conditions).

For annual average total volumes, the greatest percent reduction is for the CI, then the CLID, PP, BR, and finally RH. In terms of annual average mean flows, the greatest percent reduction is seen with the CLID model, followed by the CI and PP models (shown to be the same statistically, $p>0.05$ and t Stat < t Critical two tail), the BR model, and finally the RH model. For the annual average peak flows, the greatest percent

Table 3. Percent Reductions from the NC Model for the Duration of the Long-Term Precipitation Record

Model	Average % Reduction of Annual Average Volume	Average % Reduction of Annual Average Mean Flows	Average % Reduction of Annual Average Peak Flows
CI	99%	65%	96%
RH	34%	31%	32%
BR	62%	58%	62%
PP	67%	59%	66%
CLID	97%	94%	96%

reduction is seen with the CI and CLID models (shown to be statistically the same, p>0.05 and t Stat < t Critical two tail), the PP model, the BR model, and finally the RH model. These results will be further broken down and discussed in the following sections. The statistical analyses are all included in Appendix F.

Cumulative Volume

The total cumulative volume of outflow for the entire duration (60 years) for each of the models can be seen in Figure 3 with exact totals shown in Table 4. All of the post-development models saw a reduction in outflow volumes when compared to the NC model. These reductions in outflow from the models, when compared to the NC model, were all shown to be statistically significant. Only the NC and RH models were compared for significance because these were the closest in total volume, since they proved to be statistically different, the rest will be as well. The CI and CLID models were also tested against the P model. They both proved to be statistically different from the P model. This means that centralized infiltration decreases total volume of outflow more than even the predevelopment conditions.

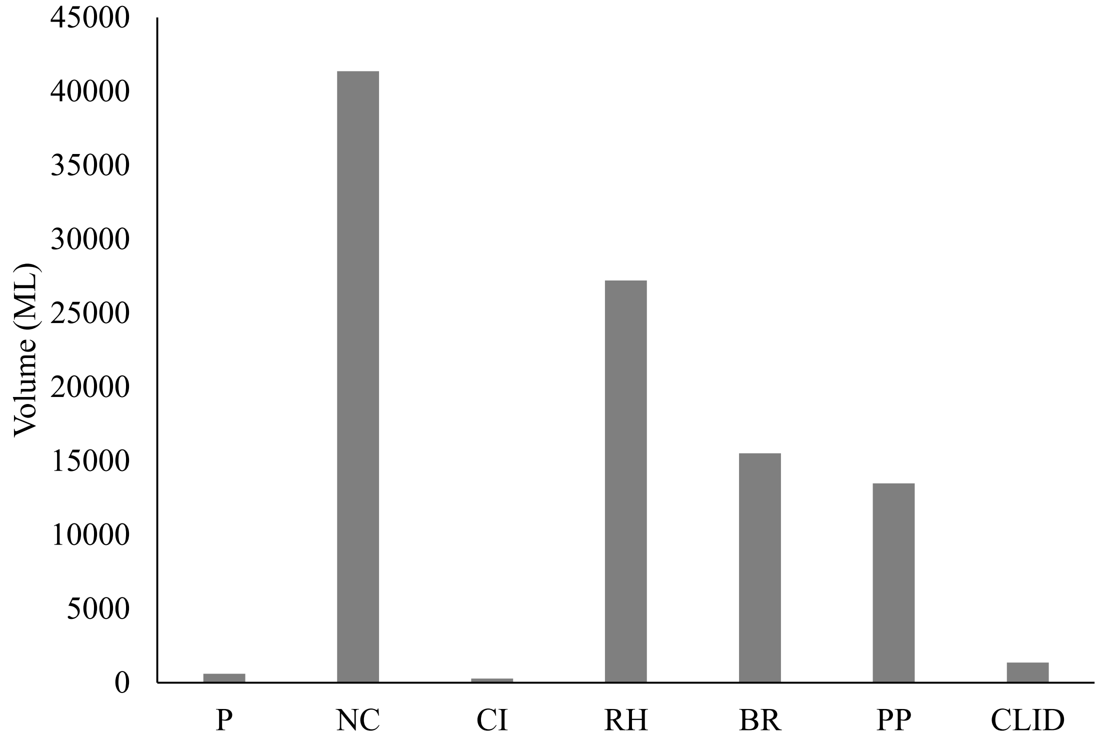

Figure 3. Cumulative Volume of Outflows (ML) for the Long-Term Data Set

Table 4. Cumulative Volumes of Outflow with Long-Term % Reductions

Model	Total Volume of Outflow (ML)	% Reduction in outflow volume
P	592	99%
NC	41,361	--
CI	263	99%
RH	27,197	34%
BR	15,514	63%
PP	13,487	68%
CLID	1,358	97%

Average Total Annual Volume

The average yearly total volumes are shown in Figure 4. Visually the average yearly total volumes are very similar to the cumulative total volumes. Once again the RH model proved to be statistically different from the No Controls model ($p<0.05$ and t Stat > t Critical two tail). All of the models were tested, and they all proved to be statistically different from one another ($p<0.05$ and t Stat > t Critical two tail). Thus, we can see that CI provides the greatest percent of reduction of outflow volume, followed by CLID, PP, BR, and finally RH (Table 5).

Average Total Annual Volumes by Type of Year

All of the outflow volumes were divided by the type of precipitation year (Figure 5). Each of the models was shown to generate outflows that are statistically different between wet and dry years ($p<0.05$ and t Stat > t Critical two tail). By calculating average percent reductions for each of the models, it was possible to compare the performance of each model in wet and dry years as well (Table 6). Every model, except for the RH model, proved to be statistically different ($p<0.05$ and t Stat > t Critical two tail) based on the percent reductions in outflow that it produced. Based upon this it is possible to say that CI performs better in dry years while BR, PP, and CLID perform better in wet years. RH performs equally well in both wet and dry years for reducing outflow volumes.

Exceedance Probability of Annual Volumes

The average annual outflow volume exceedance over the 60-year precipitation range was plotted for all simulations (Figure 6). The exceedance probability is the probability that a value will be exceeded, in this case the value is the outflow volume of a

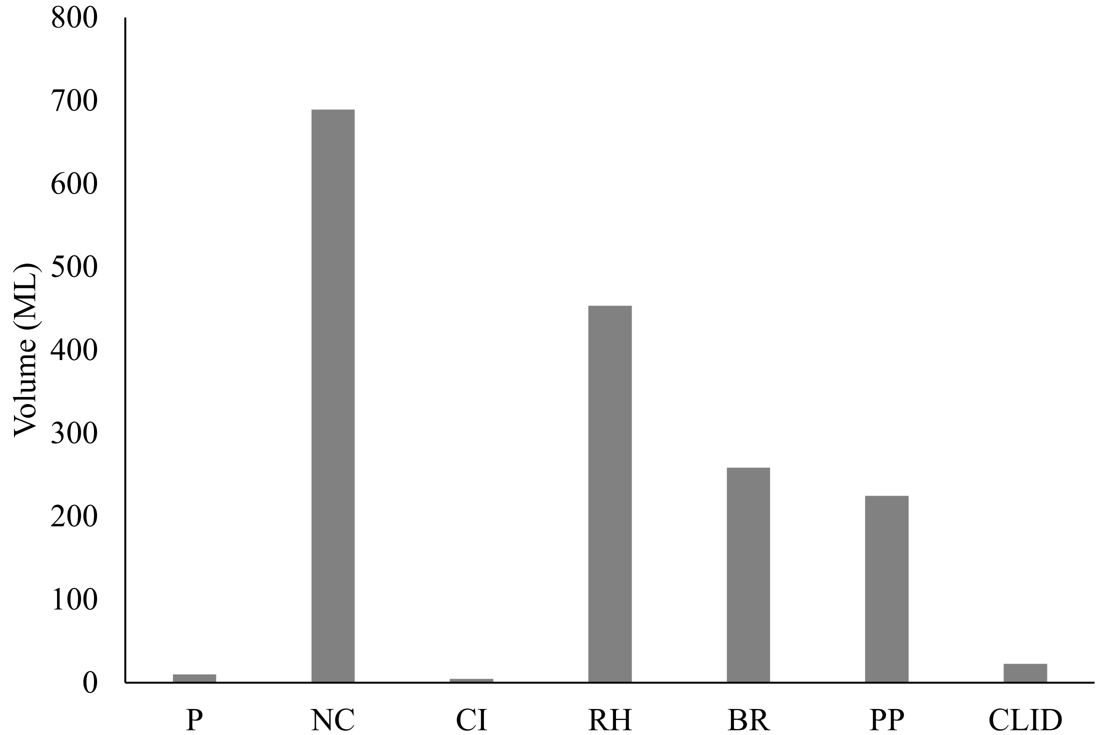

Figure 4. Annual Average Total Volumes of Outflow (ML)

Table 5. Annual Average Outflow Volume and Corresponding % Reduction (All Years)

Model	**Annual Average Outflow Volume (ML)**	**Average % Reduction of Annual Average Volume**
P	689	99%
CI	4	99%
RH	453	34%
BR	259	62%
PP	225	67%
CLID	23	97%

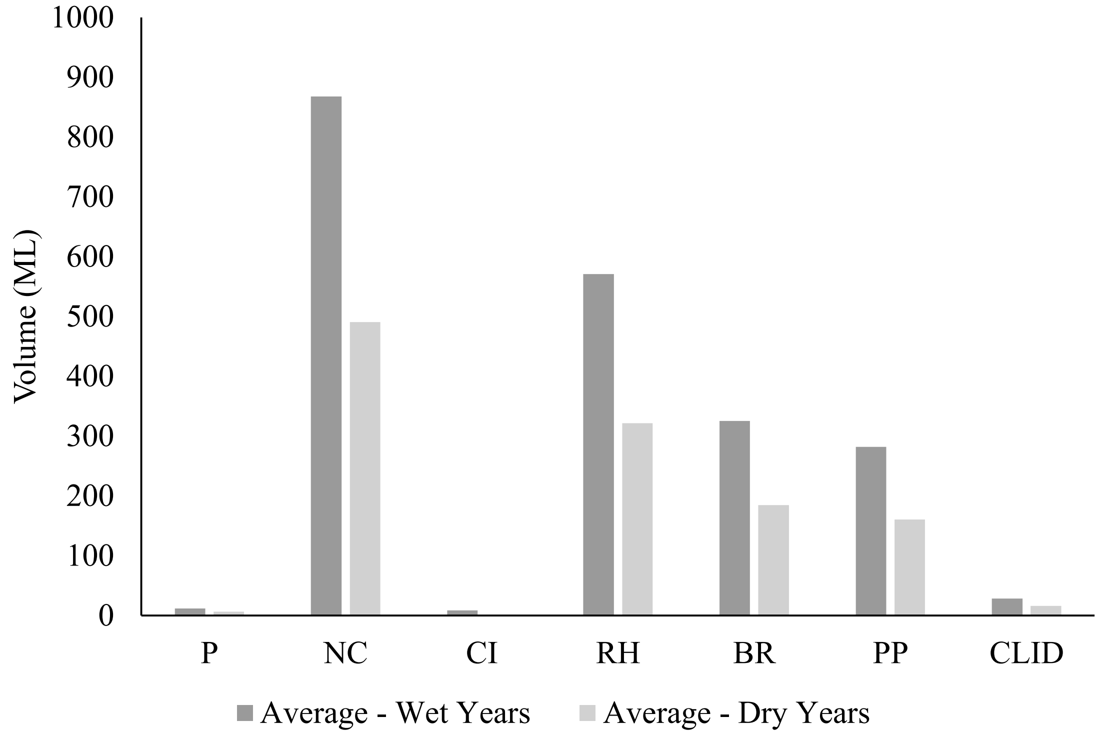

Figure 5. Annual Volumes of Outflow by Type of Year

Table 6. Average % Reduction of Annual Volumes (Wet vs. Dry Years)

	Average % Reduction of Annual Volume	
Model	**Wet Years**	**Dry Years**
CI	99%	100%
RH	33%	33%
BR	63%	62%
PP	68%	67%
CLID	97%	97%

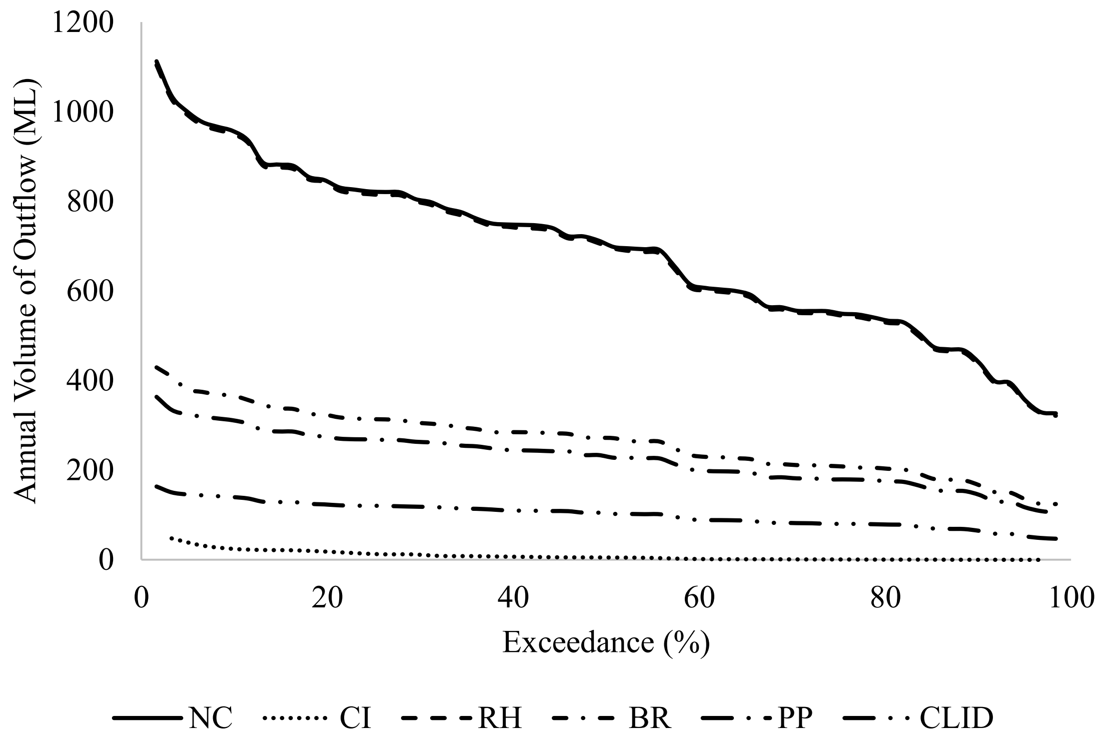

Figure 6. Exceedance Probability of Average Annual Runoff Volume

given model. Based on this figure it is possible to see the probability of outflow volume that a model will expect. Once again, the NC and RH models show the greatest outflow volume with the largest range of possible outflow volumes based on exceedance probability. The BR model is the third, followed by PP, CLID, and finally the CI model. These last four models also show a small range of possible outflow volumes based on exceedance probability.

Average Annual Mean Flows

The average of all years for annual mean flows is shown in Figure 7. Doing a statistical analysis of these values revealed that the Predevelopment and CLID models have mean flows that are statistically the same ($p > 0.05$ and t Stat < t Critical two tail).

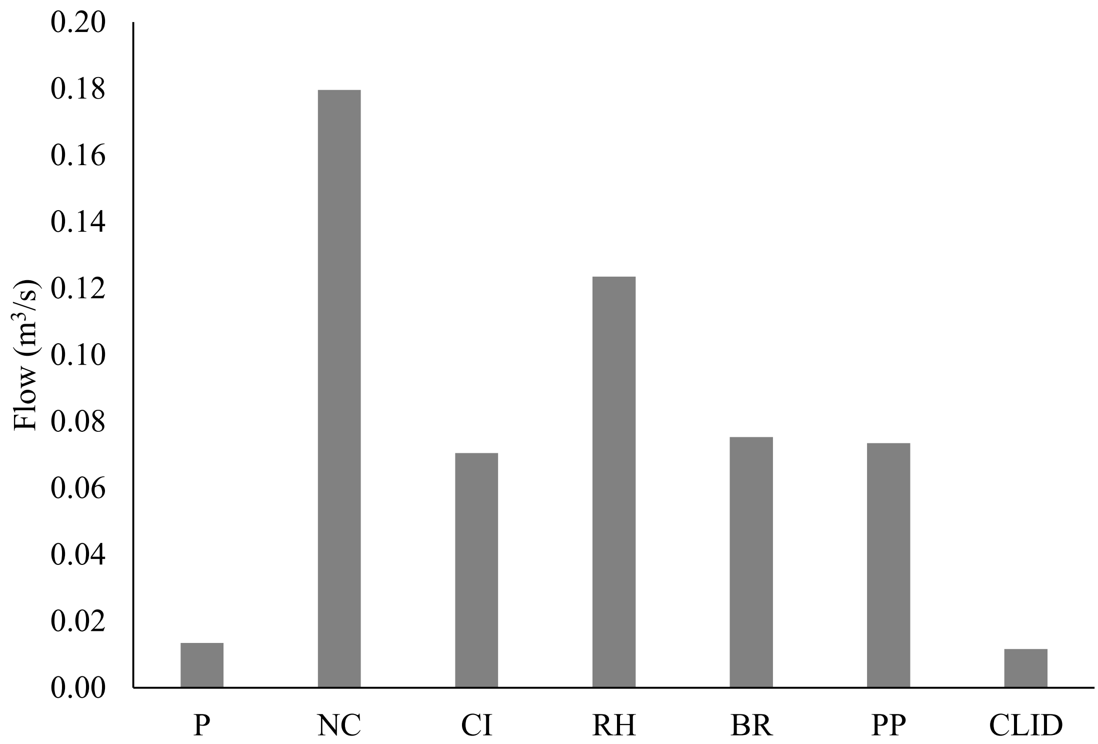

Figure 7. Average Annual Mean Flows

In addition, the mean flows of the CI, BR, and PP models are statistically the same ($p > 0.05$ and t Stat < t Critical two tail). The rest of the models are statistically different ($p < 0.05$ and t Stat > t Critical two tail). Thus in terms of model mean outflows, CLID offers the greatest reduction, CI/BR/PP are second, and RH offers the least reduction in annual mean outflows (Table 7).

Average Annual Mean Flows by Type of Year

Average annual mean flows were further determined for every wet and dry precipitation years. This average mean flow is compared in Figure 8 for each SWMM model. A statistical analysis of the mean flows between the year types shows that each

Table 7. Average % Reduction of Mean Flows (All Years)

Model	Average % Reduction of Mean Flows (All Years)
P	93%
CI	65%
RH	31%
BR	58%
PP	59%
CLID	94%

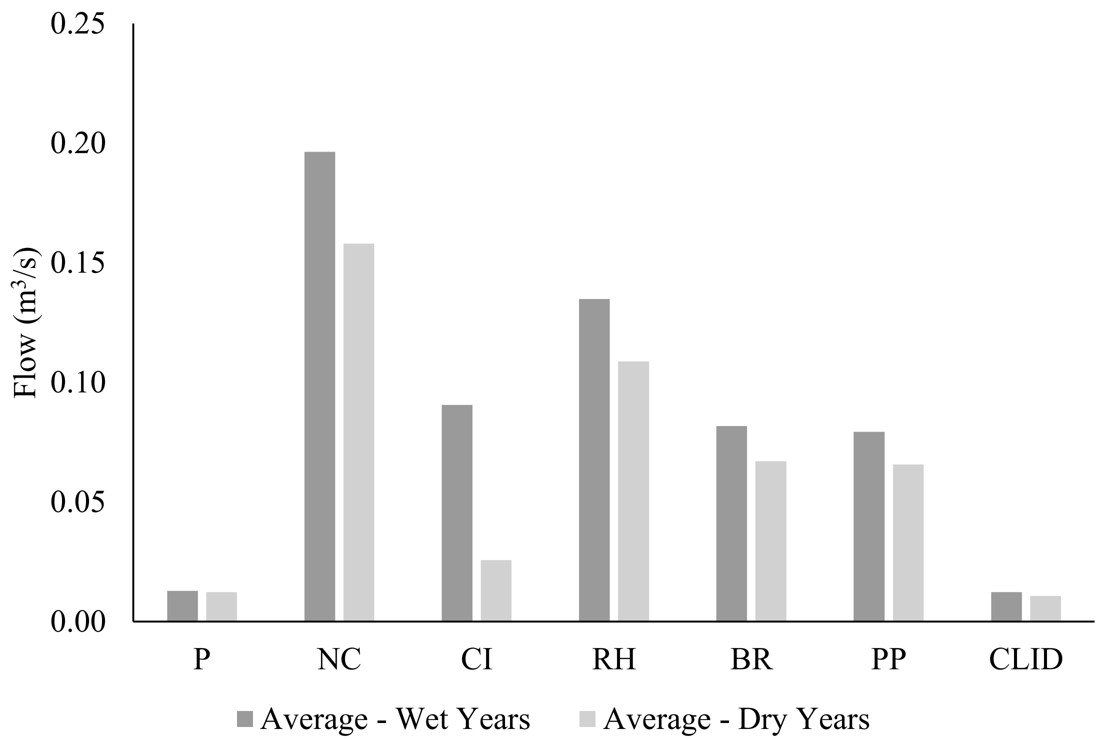

Figure 8. Average of the Annual Mean Flows by Type of Year

model performs differently in wet and dry years based upon the type of precipitation it receives ($p < 0.05$ and t Stat > t Critical two tail).

By calculating average percent reductions for each of the models, it was possible to further compare the performance of each model in wet and dry years. A statistical analysis of significance of percent reductions was conducted for each model between the wet and dry years, this revealed that each model, except for the RH, showed a statistically difference ($p < 0.05$ and t Stat > t Critical two tail). Thus, it can be seen that the CI model performs better in dry years, the BR, PP, and CLID models perform better in wet years. RH did not show any significant difference in % reductions of outflow based on the type of year (Table 8).

Exceedance Probability of Annual Mean Flows

The average annual mean outflow exceedance over the 60-year precipitation range was plotted for all simulations (Figure 9). Based on this figure it is possible to see the probability of mean outflow, in m^3/s, which a model will expect. All models, except for the CI model, show a small range of possible outflow volumes based on exceedance probability. This means that the mean outflow from any of these models should remain pretty steady no matter the type of precipitation event. The CI model does show a large

Table 8. Average % Reduction of Mean Flows (Wet vs. Dry Years)

	Average % Reduction of Mean Flows	
Model	**Wet Years**	**Dry Years**
CI	55%	86%
RH	30%	30%
BR	58%	58%
PP	60%	58%
CLID	94%	93%

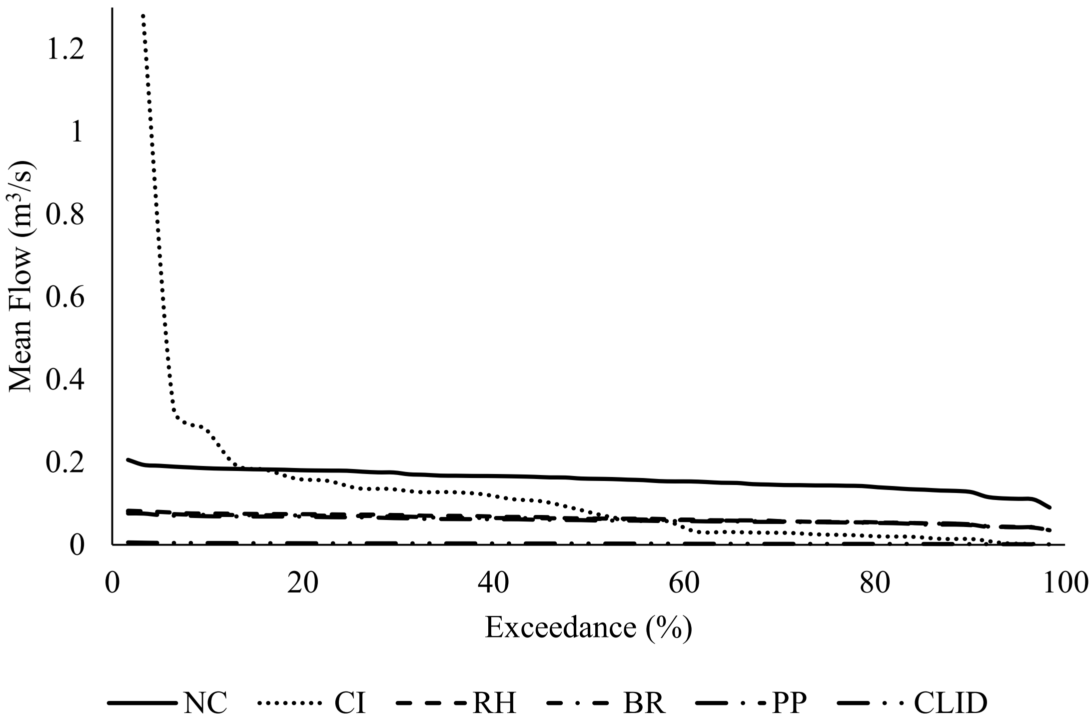

Figure 9. Exceedance Probability for Average Annual Mean Outflows

range of variability, and the model is expected to have a higher mean outflow for a 100-year precipitation event versus a 1-year precipitation event.

Average Annual Peak Flows

The average annual peak flows are shown in Figure 10. A statistical analysis compared the means of these SWMM models. All of the models proved to be statistically different ($p < 0.05$ and t Stat > t Critical two tail) from one another except for the Predevelopment, CI, and CLID models, which were statistically the same ($p > 0.05$ and t Stat < t Critical two tail). This corresponds to the percent reductions in peak flows, thus it is clear that CI and CLID are able to reduce peak flows to predevelopment levels (Table 9). Then PP is the second most effective, followed by BR, and finally RH.

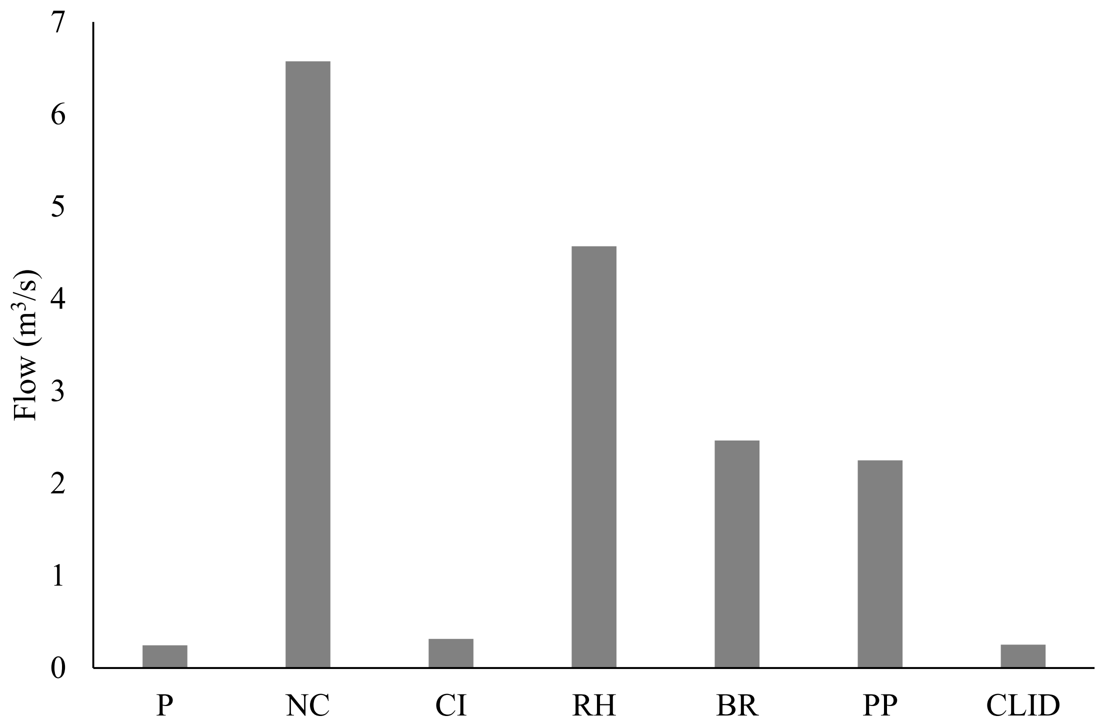

Figure 10. Annual Average Peak Flows

Table 9. Average % Reduction of Peak Flows (All Years)

Model	Average % Reduction of Peak Flows (All Years)
P	98%
CI	96%
RH	32%
BR	62%
PP	66%
CLID	96%

Average Annual Peak Flows by Type of Year

The average peak flows for RH, BR, PP, and CLID based on the type of year were all statistically the same, i.e., there is no statistical difference between their wet and dry years ($p > 0.05$ and t Stat < t Critical two tail). The only LID scenario with statistically different ($p < 0.05$ and t Stat > t Critical two tail) wet and dry years is the CI model (Figure 11).

By calculating average percent reductions for each of the models, it was possible to further compare the performance of each model in wet and dry years. Once again a statistical analysis was conducted to determine the significance between the percent reductions between years. The only model that proved to be statistically significant for percent reductions between years was the CI model ($p < 0.05$ and t Stat > t Critical two tail), which performs better in dry years. The rest of the models did not show any significant difference ($p > 0.05$ and t Stat < t Critical two tail) in percent reductions of outflow based on the type of year (Table 10). This shows that the RH, BR, PP, and CLID models all perform equally well in wet and dry years for average annual peak flow reductions, and the CI model will perform better in dry years.

Exceedance Probability of Annual Peak Flows

The average annual peak outflow exceedance over the 60-year precipitation range was plotted for all simulations (Figure 12). Based on this figure it is possible to see the probability of peak outflow, in m^3/s, which a model will expect. All models show a range of possible outflow volumes based on exceedance probability. This means that the mean outflow from any of these models should be expected to have higher peak outflows resulting from a 100-year precipitation event versus a 1-year precipitation event.

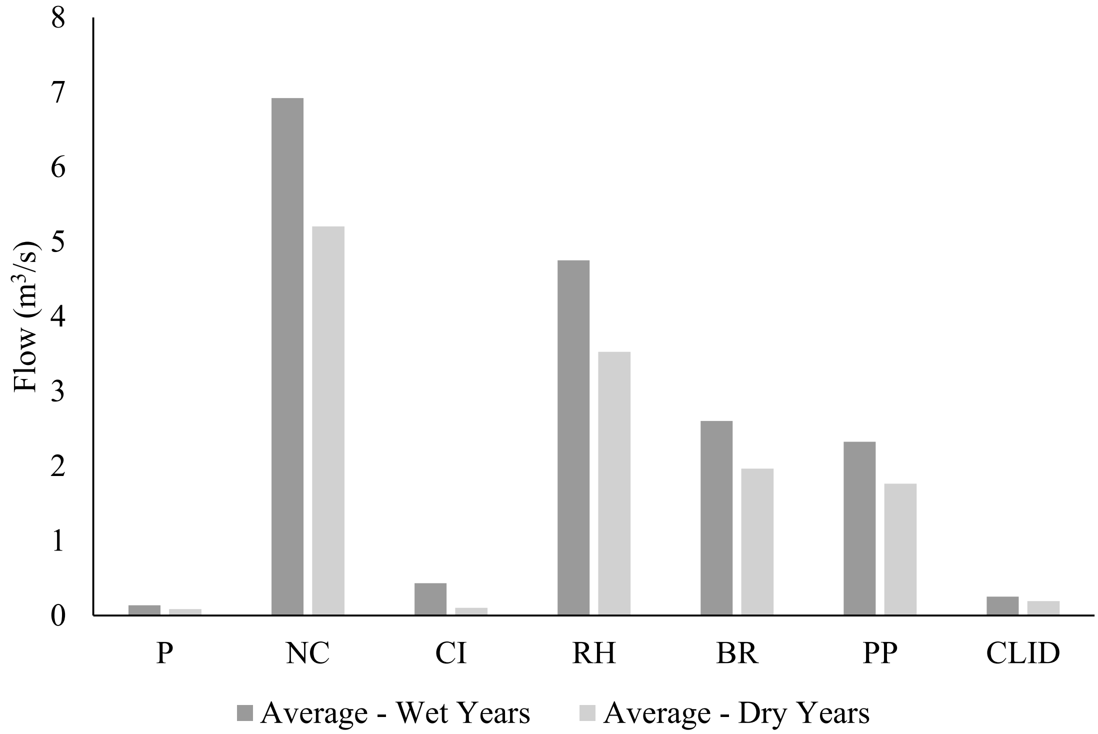

Figure 11. Average of the Annual Average Peak Flows by Type of Year

Table 10. Average % Reduction of Peak Flows (Wet vs. Dry Years)

	Average % Reduction of Peak Flows	
Model	**Wet Years**	**Dry Years**
CI	93%	98%
RH	31%	33%
BR	62%	62%
PP	66%	66%
CLID	96%	96%

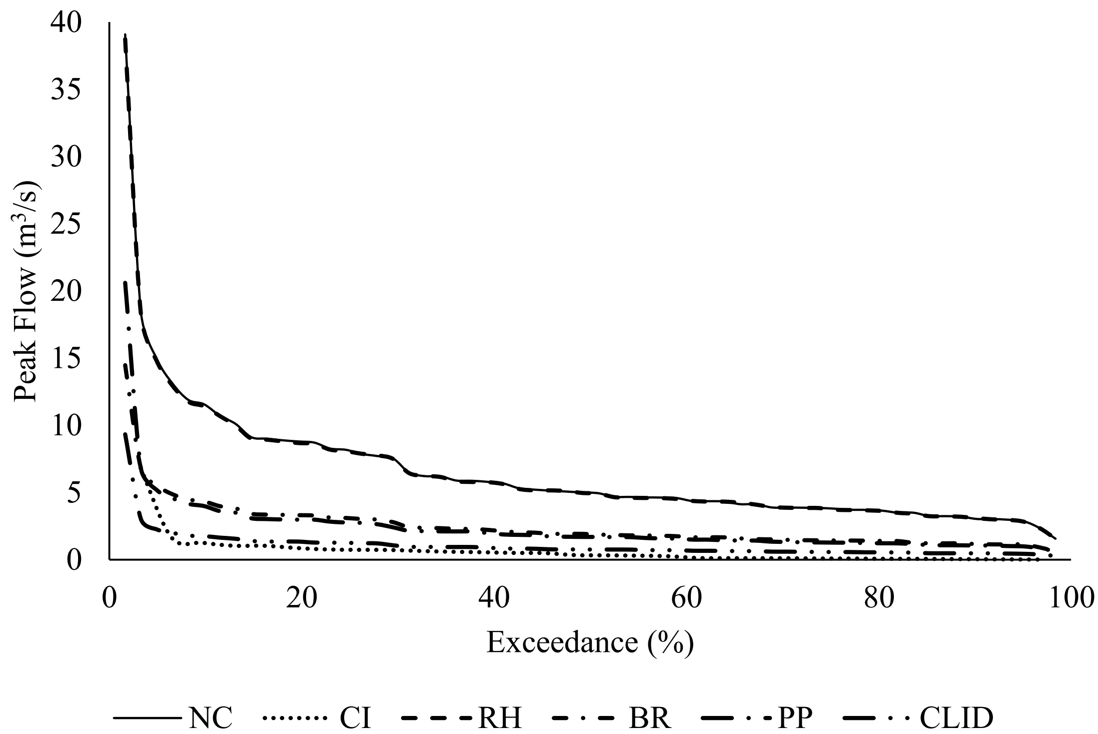

Figure 12. Exceedance Probability of Average Annual Peak Flows

Long-Term Results Summary

To determine the significance of these results, their effectiveness of decrease in volume of outflow per m^2 of impervious area treated was calculated (Table 11). This normalized the results to show the decrease in outflow volume that can be expected based on the type of stormwater management scenario implemented for the same amount of impervious area. In comparison to previously presented results, the CLID model proved to decrease the volume of outflow the most. The second most effective was the large infiltration basins (CI model), followed by the PP model, the BR model, and finally the RH model. It is important to reiterate that these results are for watershed implementation of the stormwater management techniques.

Table 11. Decrease in Volume per m^2

Stormwater Management	Decrease in Volume of Outflow (L) per m^2 of impervious area treated
CI	252
RH	87
BR	159
PP	183
CLID	262

Design Storm SWMM Model Results

Design storms were also utilized in order to verify the hydrological effects of the various SWMM models. The results of the design storms are provided in Table 12, Table 13, and Table 14. The continuity errors are low for all of the models, less than 1%. The hydrographs are shown below for each model for each design storm (Figure 13, Figure 14, and Figure 15). For the 2-year 24-hour storm, the peak discharges range from 0 m^3/s to 5.4 m^3/s. For the 10-year 24-hour storm, the peak discharges range from 0 m^3/s to 8.3 m^3/s. For the 25-year 24-hour storm, the peak discharges range from 0 m^3/s to 11.6 m^3/s. The NC model experiences the longest durations of runoff with the greatest peak discharges and the CI model generates no peak discharges due to the infiltration basins capturing all of the runoff. The RH model shows similar peak discharges and runoff as the NC model. This was expected since rain barrels are not heavily implemented in the model. The BR, PP, and CLID are progressively more effective in reducing runoff and peak discharge, which confirms the long-term simulation results. None of these design storm model results are able to meet the predevelopment levels, however, the CI results did exceed them.

Table 12. 2-year 24-hour Design Storm

Model	Runoff Continuity Error (%)	Flow Routing Continuity Error (%)	Runoff (mm)	Peak Discharge (m^3/s)	Duration (hours)
P	-0.089	0.000	0.0	0.01	0.5
NC	-0.098	0.000	6.6	5.4	20.5
CI	-0.091	0.001	7.5	0.0	0.0
RH	-0.098	0.000	6.6	5.2	19.5
BR	-0.100	0.000	2.4	2.2	17.5
PP	-0.125	0.000	2.1	2.1	17.5
CLID	-0.122	0.000	0.8	0.9	12.0

Table 13. 10-year 24-hour Design Storm

Model	Runoff Continuity Error (%)	Flow Routing Continuity Error (%)	Runoff (mm)	Peak Discharge (m^3/s)	Duration (hours)
P	-0.075	0.000	0.1	0.2	1.0
NC	-0.103	0.000	10.5	8.3	24.0
CI	-0.094	0.002	11.6	0.0	0.0
RH	-0.102	0.000	10.4	8.1	23.5
BR	-0.102	0.000	3.8	3.3	21.0
PP	-0.106	0.000	3.3	3.1	21.0
CLID	-0.107	0.000	1.4	1.4	16.0

Table 14. 25-year 24-hour Design Storm

Model	Runoff Continuity Error (%)	Flow Routing Continuity Error (%)	Runoff (mm)	Peak Discharge (m^3/s)	Duration (hours)
P	-0.070	0.000	0.2	0.6	1.5
NC	-0.110	0.000	14.7	11.6	25.5
CI	-0.100	0.001	16.0	0.0	0.0
RH	-0.110	0.000	14.6	11.3	25.0
BR	-0.097	0.000	5.3	4.6	23.5
PP	-0.109	0.000	4.7	4.3	23.0
CLID	-0.104	0.000	2.0	1.9	17.5

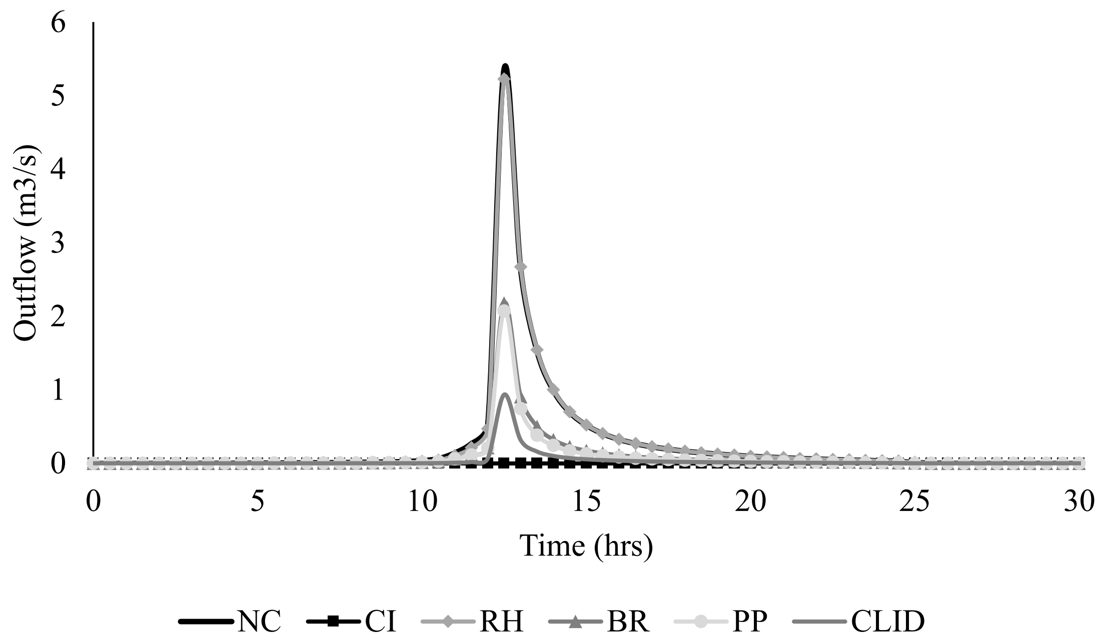

Figure 13. 2-year 24-hour Design Storm Hydrograph

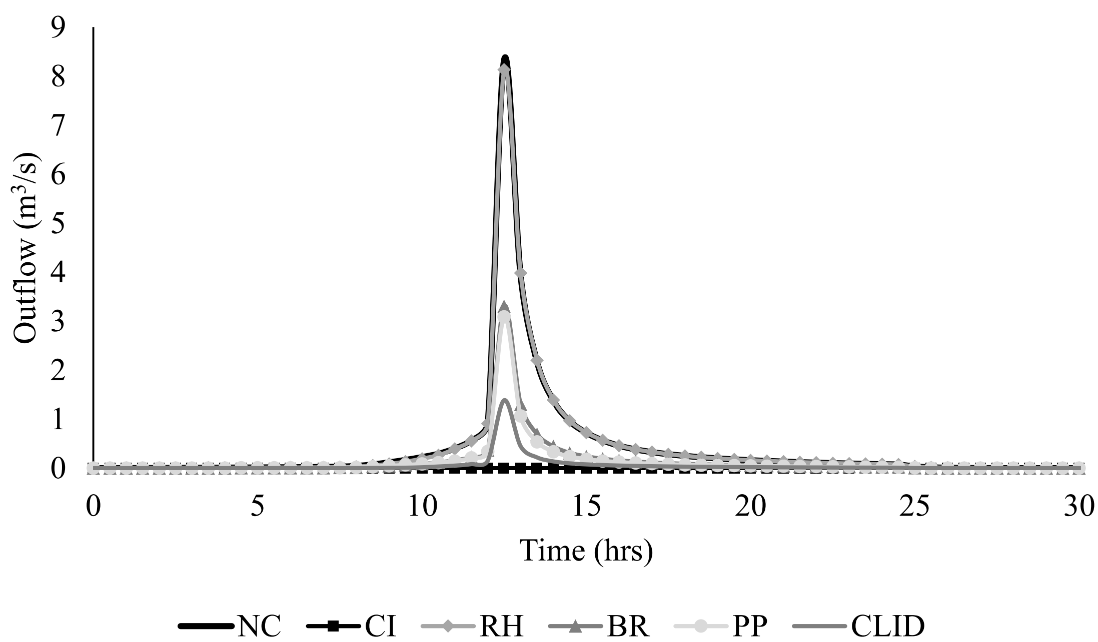

Figure 14. 10-year 24-hour Design Storm Hydrograph

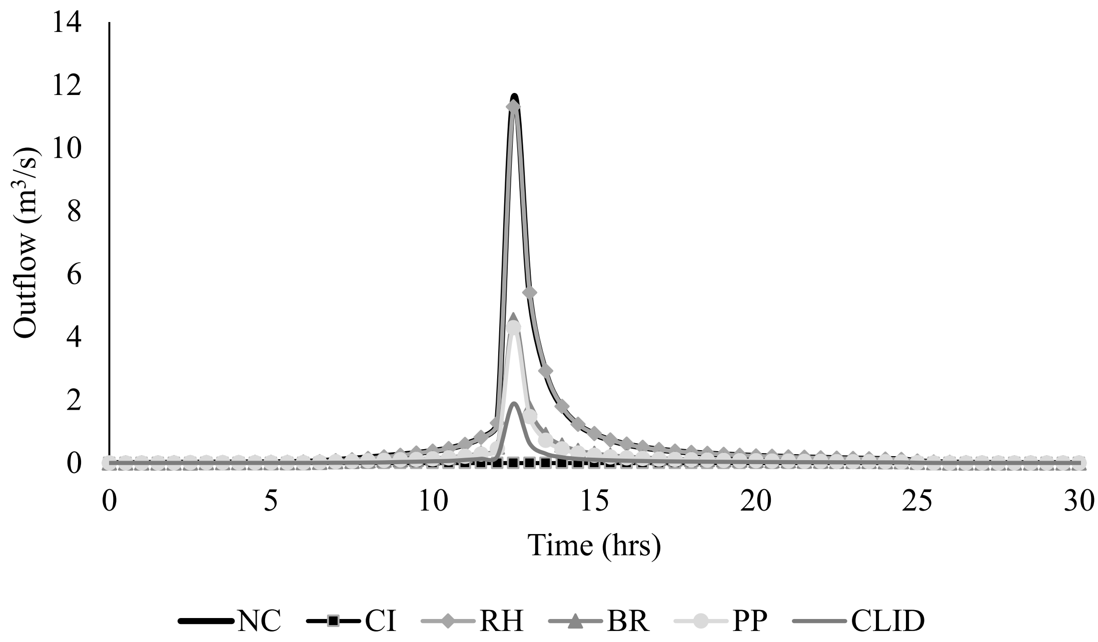

Figure 15. 25-year 24-hour Design Storm Hydrograph

Cost Estimates

Another important element of the performance analysis of LID implementation is its cost. Even if a design idea provides excellent reductions in outflow, peak flows, and mean flows, it needs to be able to be implemented at a reasonable cost.

Cost estimates for each of the modeled scenarios were completed. Capital costs are the costs incurred for the initial installation and build, and maintenance costs consist of maintenance issues that must be taken care of on a timely basis to ensure the proper function of the stormwater management technologies (Table 15).

In order to be able to make appropriate decisions about what type of stormwater management to implement, it is necessary to be aware of the capital costs to build, as well as the maintenance costs. These costs were combined into one number to be visually easier and represent the whole life costs (Table 16). These values represent the costs of

Table 15. Capital and Maintenance Costs

Stormwater Management	Capital Costs	Maintenance Costs (50 years)
Rain Barrels	$ 532,000.00	$ 1,261,000.00
Porous Pavement	$15,861,000.00	$12,029,000.00
Differential Cost of Porous Pavement (Subtracting Cost of Regular Asphalt)	$13,421,000.00	$10,168,000.00
Bioretention	$ 1,997,000.00	$ 825,000.00
Differential Cost of Bioretention (Subtracting Cost of Regular Landscaping)	$ 1,450,000.00	$ 819,000.00
Comprehensive LID*	$15,403,000.00	$12,247,000.00
Large Infiltration Basins	$16,874,000.00	$ 169,000.00

*Calculated with the differential cost of porous pavement and bioretention

implementing the stormwater management systems as they are represented in the SWMM models, they are not able to be directly compared to one another, however, due to their differing levels of implementation within each of the models.

From these values an overall cost effectiveness was calculated (Table 17). Since the whole life costs are for a 50-year period, a 50-year decrease in outflow volumes was utilized (1961-2011). Porous pavement proved to be the most expensive option of the group, followed by the CLID model, which was expected since this model incorporates porous pavement. The infiltration basins of the CI model were the third most expensive,

Table 16. Whole Life Costs

Stormwater Management	Whole Life Costs
Rain Barrels	$1,793,000.00
Porous Pavement	$27,890,000.00
Differential Cost of Porous Pavement (Subtracting Cost of Regular Asphalt)	$23,589,000.00
Bioretention	$2,822,000.00
Differential Cost of Bioretention (Subtracting Cost of Regular Landscaping)	$2,269,000.00
Comprehensive LID*	$27,650,000.00
Large Infiltration Basins	$17,043,000.00

*Calculated with the differential cost of porous pavement and bioretention

Table 17. Cost for a 50-year Period Based on Decrease in Outflow Volumes

Stormwater Management	Whole Life Costs ($)/Decrease in Volume over 50 years (ML)	Whole Life Costs ($)/Decrease in Volume over 50 years (1000L)
CI	$ 497.61	$0.50
RH	$ 151.92	$0.15
BR	$ 105.30	$0.11
PP	$1,015.54	$1.02
CLID	$ 829.45	$0.83

*Calculated with the differential cost of porous pavement and bioretention

followed by the rain barrels of the RH model, and finally the bioretention gardens proved to be the least expensive.

In addition to calculating the cost effectiveness based on whole life costs per decrease in volume of outflow, the cost of reduction per m^2 of impervious area treated was also calculated (Table 18). This cost per treatment of a m^2 of impervious area is helpful to see when cost is may be an issue. Here it is possible to see that the most cost

Table 18. Cost Per m^2 of Impervious Cover Treated

Stormwater Management	Whole Life Costs per m^2 of impervious area treated
CI	\$6.27
RH	\$0.66
BR	\$0.83
PP	\$9.27
CLID	\$10.86

*Calculated with the differential cost of porous pavement and bioretention

effective stormwater management strategy is the RH model with the use of rain barrels (\$0.66/m^2). The use of rain barrels, in the RH model, is capable of treating a m^2 of impervious area for the lowest cost compared to the other investigated options. The second least expensive alternative is the implementation of bioretention gardens in the BR model (\$0.83/m^2). The final three LID technologies that were modeled are progressively more expensive to put into practice, which is expected since they have the highest costs of implementation.

DISCUSSION

SWMM Model Results

All of the LID models showed a reduction in outflow from the No Controls model. As such, any of these options may be implemented with the goal of reducing outflows in the community. Each individual model was not expected to perform in precisely the same manner since the various LID features were not equally implemented. Thus a comparison of the LID technologies is comprehensive in this case, and does not attest do differing levels of implementation and percentage of impervious area treated for each individual low impact development technique. Overall results are discussed below and Table 19 include a summary of performance based on the precipitation year.

For volume reductions, overall the results show that CI (large infiltration basins) performed the best in reducing stormwater outflow volumes. The CI was also shown to perform better in the dry years. The CLID was a close second in volume reduction, this model performed better in wet years. Third best for reducing outflow was PP (performed better in wet years), fourth was BR (performed better in wet years), and finally RH showed the lowest percent reductions (it performed equally in wet and dry years).

For mean flow reductions, overall the results show that the CLID model performed the best in reducing stormwater mean outflows. The CLID was also shown to perform better in the wet years. The CI, PP, and BR models showed statistically the same reduction of mean outflows with the first one performing better in dry years and the second two performing better in wet years. RH showed the lowest percent reductions of

Table 19. Optimal Performance Based on Year Type

	Volume Reduction		Mean Flow Reduction		Peak Flow Reduction	
	Wet	Dry	Wet	Dry	Wet	Dry
CI		X		X		X
RH	X	X	X	X	X	X
PP	X		X		X	X
BR	X		X		X	X
CLID	X		X		X	X

mean outflows (it performed equally in wet and dry years).

For peak flow reductions, overall the results show that the CLID model and the CI performed the best in reducing stormwater peak outflows (the two models were the same statistically). The CLID was shown to perform equally in both wet and dry years, and the CI performed better in the dry years. Third best for reducing peak outflows was PP, fourth was BR, and finally RH showed the lowest percent reductions (all three performed equally well in wet and dry years). The design storms were able to verify these results.

The design storms were also able to show the durations of runoff resulting from storm events. These results indicated that the NC model, which is the developed land with no stormwater controls, produces the longest duration of runoff. The RH model only shortens the duration by 30 minutes. The BR and PP models shorten the duration by about 2-3 hours, the CLID model shortens duration by about 8 hours, and the CI model has a duration of less than 30 minutes.

There are many elements in this analysis and many ways to approach a best solution. The best solution will depend on the type of reductions that are desired and/or costs involved. It is important to note that these models were all created in a way that they would reflect realistic implementation in this particular community. In order to get a full idea of the implementation, a cost analysis of the different models was conducted in order to help quantify the use of the technologies. In terms of whole life costs ($) / decrease in volume over 50 years (1000L), BR proved to be the least expensive option, followed by RH, CI, CLID, and finally PP.

Based on all of these results (Table 20), the final recommendation is that projects should first consider bioretention and then rainwater harvesting as options for stormwater management. Porous pavement is an effective choice, however, its cost may be a deterrent. Since the porous pavement was included in the comprehensive LID model, this is the reason for the high cost of this option. Centralized infiltration is a good choice for new developments, it is very effective, and in the case of Daybreak, Utah it allows for the reduction in costs due to the lack of traditional stormwater conveyance systems. Due to the size limitations of large-scale infiltration, however, smaller decentralized LID technologies are a better choice for existing communities. Options such as rain barrels and bioretention gardens carry a lower cost, smaller land requirement, and they are effective in reducing stormwater runoff.

Internal/External Threats to Validity

The main concern with these results is the lack of calibration data. Calibration was done using the rational method due to the unavailability of data. Field performance of these stormwater practices should be further analyzed in order to validate these results. However, these results can be used at the preliminary basis for a large-scale study and they do reveal insights as to general conclusions that can be drawn for the comparison of smaller decentralized LID Technologies compared to large scale infiltration practices.

Table 20. Summary of Findings

Stormwater Model	Decrease in Volume of Outflow (L)/m^2 of impervious area treated	Annual Average of Outflow Volume (ML)	Whole Life Costs per m^2 of impervious area treated	Average % Reduction of Annual Average Volume	Average % Reduction of Annual Average Mean Flows	Average % Reduction of Annual Average Peak Flows	Whole Life Costs ($)/Decrease in Volume of Outflow over 50 years (1000L)
CLID	262	23	$10.86	97%	94%	96%	$0.83
CI	252	4	$6.27	99%	65%	96%	$0.50
PP	183	225	$9.27	67%	59%	66%	$1.02
BR	159	259	$0.83	62%	58%	62%	$0.11
RH	87	453	$0.66	34%	31%	32%	$0.15

CONCLUSION

Analysis Results

Using Daybreak, Utah as a case study has shown the viability of LID technologies in semi-arid climates. Large scale centralized infiltration was shown to be the most effective in reducing total volumes of outflow, however, the cost of this, both in terms of price and land required, remains high. In addition, this model does not work well in order to decrease peak discharges and maintain steady mean outflows. Other smaller-scale decentralized methods, such as bioretention, rainwater harvesting, and porous pavement were proven to have a positive effect on reducing the effects of urbanization in terms of volume of outflow, mean outflows, and peak discharges. In addition, any of these technologies are easier to retrofit in already developed communities (compared to trying to incorporate large scale infiltration basins) and they can come at a lower cost.

Knowing how to decrease the effects of urbanization should be encouraged in new developments and for retrofitting. These results show that it would be possible to incorporate small scale LID technologies into a community to match the results of building large centralized infiltration basins. If such a large financial expenditure is not possible, it is still viable to retrofit smaller scale, less expensive option to make an impact in decreasing stormwater runoff, decreasing peak flows, and decreasing mean flows.

Future Research

This study was done using historical precipitation data going back approximately 60 years. A future investigation of the use of LIDs should utilize future climate predictions in order to see the effects that a changing climate may have on the hydrological response of LID technology. The comparison of the technologies in wet vs. dry precipitation years does show some indications of which technologies will perform better. Based on the results of this paper, if the climate in Utah becomes wetter, the elements in the comprehensive LID model, porous pavement, and bioretention will perform better. Centralized infiltration using large-scale infiltration basins will perform better in dry years. Rainwater harvesting, using rain barrels, should perform equally well no matter what the future climate holds.

More investigations could also center on the effects of increasing infiltration through the use of LIDs in Utah and the semi-arid western United States. Heiberger (2013) began this investigation, but more research needs to be conducted. It would be interesting to be able to compare the effects of infiltration by smaller scale LIDs and large scale infiltration basins to provide another point of comparison.

In addition, this thesis did not analyze any of the potential pollutant loads within this area. Further investigations could also analyze the impacts of the various LID techniques on pollutant loads and their reductions. This could be used as an additional parameter for consideration for choosing various stormwater management practices.

APPENDIX A

PRECIPITATION ANALYSIS

Precipitation Analysis

Figure 16 shows the exceedance frequency plot of total event precipitation for the long-term continuous precipitation dataset. Approximately 90% of the precipitation events are less than or equal to 10 mm and approximately 75% are less than or equal to 5 mm in total event depth.

Precipitation by Year Type

The precipitation data were analyzed in order to determine the type of year based on precipitation. This would be further used in order to try to determine trends. Based on the data, an average year was between 330 – 432 mm of rain, a dry year received less than 330 mm of rain, and a wet year received more than 432 mm of rain. Average precipitation by type of year is listed in Table 21.

These separated years were compared to the Palmer Drought Severity Index (PDSI) to ensure that they were properly categorized. There appears to be no statistical difference between the categorization based solely on precipitation amounts and the classification based upon the PDSI (Table 22).

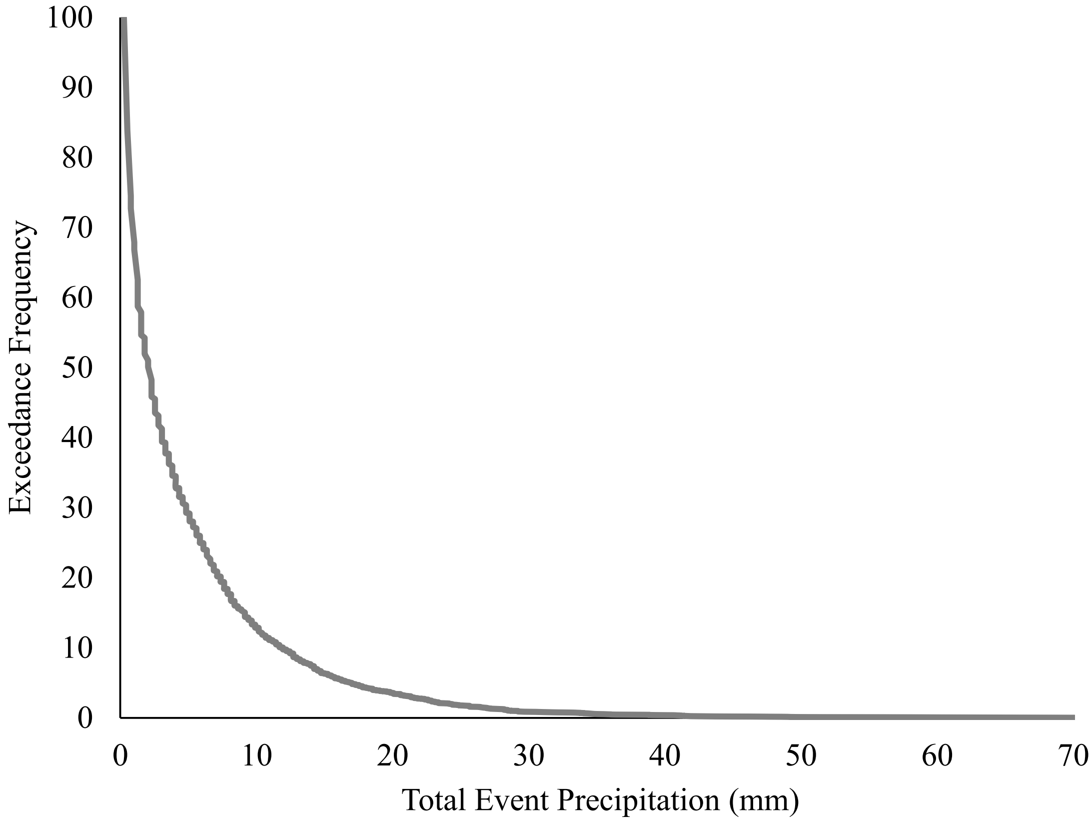

Figure 16. Exceedance Plot of Total Event Precipitation

Table 21. Average Precipitation by Type of Year

	Year Type		
	DRY	**AVERAGE**	**WET**
Average Precipitation (mm)	286.0	388.6	490.2
Standard deviation	36.0	33.2	45.3

Table 22. Palmer Drought Severity Index Classification

Palmer Classifications	
4.0 or more	extremely wet
3.0 to 3.99	very wet
2.0 to 2.99	moderately wet
1.0 to 1.99	slightly wet
0.5 to 0.99	incipient wet spell
0.49 to -0.49	near normal
-0.5 to -0.99	incipient dry spell
-1.0 to -1.99	mild drought
-2.0 to -2.99	moderate drought
-3.0 to -3.99	severe drought
-4.0 or less	extreme drought

APPENDIX B

GIS FILES

All files were obtained from the Utah Automated Geographic Reference Center (AGRC), accessed at www. http://gis.utah.gov/. The specific files utilized include: 12TVK100860, 12TVK100880, 12TVK100900, 12TVK120860, 12TVK120880, 12TVK120900, 12TVK140860, 12TVK140880, 12TVK140900, 12TVK160860, 12TVK160880, and 12TVK160900.

Contour datasets downloaded from the AGRC website were used to identify potential runoff flow paths during the development of the watershed delineation. The 2-meter contours dataset for the Salt Lake County area were used to obtain topographic information for the project study area. The shape tiles used for this study include: (12TVK100860, 12TVK100880, 12TVK100900, 12TVK120860, 12TVK120880, 12TVK120900, 12TVK140860, 12TVK140880, 12TVK140900, 12TVK160860, 12TVK160880, 12TVK160900). These tiles cover the area including and surrounding the watershed area.

The aerial photographs used for this study to identify geomorphologic changes and land cover were downloaded from the AGRC website. Specific tif tile names used are: (12TVK100860, 12TVK100880, 12TVK100900, 12TVK120860, 12TVK120880, 12TVK120900, 12TVK140860, 12TVK140880, 12TVK140900, 12TVK160860, 12TVK160880, 12TVK160900). The collection of aerial photos used for the study was taken in 2012. These tiles cover the same spatial footprint as the contour data described in the previous section.

APPENDIX C

SWMM MODEL PARAMETERS

Evaporation

Daybreak is in the Jordan River Watershed and water surface evaporation in the valley averages 42 inches per year (Salt Lake County website). Monthly evaporation data were obtained from NOAA Technical Report National Weather Service 34 (NOAA 1982). The SWMM model's calculation of evaporation based upon the climate files was chosen as the final source of evaporation data since it would allow for temperature variability in each individual year.

Manning's n and Depression Storage Parameters

The parameters for Manning's n and depression storage were determined from two well used sources. The "N Imperv" and "N Perv" parameters are typical values of Manning's n for overland flow over the impervious and pervious portion of the subcatchment, respectively (McCuen 1996). The "Dstore Imperv" and "Dstore Perv" parameters are typical values of depth of depression storage on the impervious and pervious portion of the subcatchment, respectively (ASCE 1992). These parameters were determined for the predevelopment model and for the developed models, all of which were assumed to have the same values (Table 23).

Table 23. Manning's n and Depression Storage Parameters

Model Type	**N-Imperv**	**N-Perv**	**D store-Imperv (mm)**	**D store-Perv (mm)**
Predevelopment	0.013	0.15	1.27	3.81
Developed	0.013	0.15	1.27	2.54

Soil

The Green-Ampt equation in SWMM was selected for determining infiltration rates of the soil within Daybreak. The Green-Ampt parameters utilized in the SWMM model were determined based upon the soil data from the United States Department of Agriculture (USDA) National Resources Conservation Service (NRCS) Division. The soils dataset was downloaded from the Utah Automated Geographic Reference Center (AGRC) website (http://gis.utah.gov/), which contains the soil profiles for the entire state of Utah. The soil data were collected and are maintained by a division of USDA called the National Cooperative Soil Survey. Surveys for the Salt Lake Area were originally completed in 1899 (valley west of Jordan River only) and 1974 (entire Salt Lake area). A review of the metadata for the soil data layer used for this study (i.e., the Salt Lake area) indicates the data were last updated in February 2010 (USDA 2012).

Using the soil data in GIS, it was possible to determine the percentage of each type of soil in each subwatershed. Then using these data, it was possible to calculate weighted average values for the soil parameters to use for each subwatershed. See Table 24 for the final soil parameters utilized in the SWMM models. Unfortunately this calculation is not able to account for any of the imported soils that may have been brought in during construction. Thus the soil parameters do not account for the effects of construction and land development.

Slope

The average slopes for each of the individual subcatchments were determined using GIS. These values were then verified using Google Earth Pro. This was done to ensure that the slopes being calculated by GIS did in fact reflect the slope for the most

Table 24. SWMM Parameters by Subwatershed

Daybreak Subwatershed	Total Area (ha)	Suction (mm/hr)	HydCon (mm/hr)	SMD Max (mm)	% Slope
DaybreakPkwy	14.15	100.03	11.33	8.495	4.4
EastlakeVillage_1	11.60	95.67	12.28	8.692	7.5
EastlakeVillage_2	11.95	62.94	9.35	6.221	4.0
EastlakeVillage_3	7.62	126.78	5.28	7.177	3.5
EastlakeVillage_4	5.49	58.92	7.90	5.488	2.2
EastlakeVillage_5	5.21	59.32	8.81	5.864	2.5
EastlakeVillage_6	2.55	47.37	7.04	4.682	4.5
EastlakeVillage_7	18.35	74.05	9.71	6.811	5.0
EastlakeVillage_Elementary	27.39	80.25	6.93	5.961	2.0
EastlakeVillage_North	22.01	73.34	7.93	6.079	3.0
EastlakeVillage_Temple_North	15.41	57.16	8.49	5.651	6.0
EastlakeVillage_Temple_South	22.87	76.39	11.55	7.485	7.1
FoundersParkVillage_1	3.96	58.00	8.62	5.734	2.2
FoundersParkVillage_2	2.80	56.79	8.44	5.614	3.6
FoundersParkVillage_3	1.61	72.78	10.81	7.195	4.3
FoundersParkVillage_4	10.85	74.36	10.29	7.053	1.9
FoundersParkVillage_5	2.94	73.75	10.96	7.291	3.3
FoundersParkVillage_6	2.03	58.01	8.62	5.735	4.4
FoundersParkVillage_7	13.51	70.15	9.80	6.689	3.5
FoundersParkVillage_Daybreak Elementary_East	9.88	157.78	7.49	9.292	3.4
FoundersParkVillage_Daybreak Elementary_West	17.07	162.23	7.12	9.324	3.2
FoundersParkVillage_East	17.72	113.45	11.17	8.968	2.7
FoundersParkVillage_South	48.92	123.94	8.39	8.290	1.7
GardenPark	23.72	111.94	11.15	8.900	2.0
NorthShoreVillage_1	26.70	142.61	6.53	8.300	1.6
NorthShoreVillage_2	4.72	97.16	11.77	8.552	11.1
NorthShoreVillage_3	2.56	89.72	9.58	7.387	10.9
NorthShoreVillage_4	4.31	46.21	6.06	4.249	2.9
NorthShoreVillage_5	8.27	93.63	12.60	8.736	8.8
SouthStation_SodaRow	47.04	73.34	9.84	6.834	1.8

likely flow paths of each subwatershed. Google Earth Pro was chosen because of its ease of use, capacity to import the subcatchment shape files from GIS, and its ability to individually choose the best areas for the slope determination and select multiple paths from which to determine slope.

Other subwatershed parameters

Impervious percent of coverage for each subwatershed was estimated from visual inspection and a GIS verification. See Table 24 for the final parameters utilized in the SWMM models. The “Width” parameter for SWMM was determined by dividing the total subwatershed area by its maximum length of overland flow (Table 25). The number of homes was determined for each individual watershed by counting the homes and lots to be development from the GIS files (Table 25).

Table 25. Additional SWMM Parameters

Daybreak Subwatershed	**Length of overland flow (m)**	**Width (m)**	**number of homes/ buildings**	**Area of Impervious Cover (ha)**
DaybreakPkwy	876	162	5	6
EastlakeVillage_1	275	422	57	7
EastlakeVillage_2	800	149	113	7
EastlakeVillage_3	306	249	25	5
EastlakeVillage_4	569	97	24	3
EastlakeVillage_5	400	130	48	4
EastlakeVillage_6	104	244	11	1
EastlakeVillage_7	847	217	108	12
EastlakeVillage_Elementary	978	280	214	19
EastlakeVillage_North	870	253	70	15
EastlakeVillage_Temple_North	592	260	106	9
EastlakeVillage_Temple_South	931	246	120	15
FoundersParkVillage_1	374	106	43	3
FoundersParkVillage_2	279	101	23	2
FoundersParkVillage_3	241	67	10	1
FoundersParkVillage_4	785	138	66	8
FoundersParkVillage_5	328	90	10	2
FoundersParkVillage_6	228	89	14	1
FoundersParkVillage_7	865	156	28	8
FoundersParkVillage_Daybreak Elementary_East	737	134	96	7
FoundersParkVillage_Daybreak Elementary_West	937	182	134	12
FoundersParkVillage_East	877	202	120	13
FoundersParkVillage_South	1773	276	360	34
GardenPark	1186	200	176	17
NorthShoreVillage_1	995	268	111	19
NorthShoreVillage_2	251	188	24	3
NorthShoreVillage_3	287	89	18	2
NorthShoreVillage_4	281	153	37	3
NorthShoreVillage_5	480	172	45	6
SouthStation_SodaRow	1517	310	106	28

APPENDIX D

SWMM MODELS

Predevelopment Model

The predevelopment model shows the natural characteristics and hydrological responses of the land with no urbanization. Due to the land remediation completed prior to the development of the land, the predevelopment model did have to assume the state of land as it currently is, and may not be reflective of the natural state of the area prior to remediation. Thus the model was created based upon the delineated subwatersheds and their calculated variables in GIS, such as the completed soil analysis, computed percent slopes, and areas of the subwatersheds. The use of the subwatershed data, versus an average of each of these for the entire watershed, resulted in a more precise characterization of the predevelopment conditions.

No Controls

In order to showcase the effects of urbanization without any stormwater management controls, the No Controls model was developed. This model shows the currently developed land without any stormwater infrastructure or LID features.

Centralized Infiltration

This model was created to most closely reflect the Daybreak community as it presently exists. It was built upon the No Controls model with the addition of the large infiltration basins. There are no connections to the municipal stormwater infrastructure and instead stormwater is directed towards large centralized infiltration basins to capture up to the 100-year storm event. These infiltration basins were modeled as storage units in SWMM. For each individual infiltration basin the maximum depth and ponded area parameters were calculated using GIS (Table 26).

Table 26. Storage Unit Parameters for CI model

Storage Unit	Ponded (Top) Area (m^2)	Area of Bottom (m^2)	Depth (m)
DaybreakElementary	26834.3	16930.1	3.8
DaybreakPkwy_North	39085.8	19148.0	2.4
Dog_Park	8756.7	3563.6	2.4
EastlakeCommons_Field	5942.2	3599.2	2.4
EntryLoop_Path	37585.2	16442.5	2.4
FernRidgeDr_North	4580.0	1308.4	1.8
FernRidgeDr_South	7220.3	3811.0	1.8
Finch_Park	2759.6	1170.8	2.7
Firmont_Park	6313.6	3071.9	2.1
GrandvilleAve_CurrantDr	2868.3	1001.3	3.0
IronMountainDr_North	2741.5	1418.7	2.4
IronMountainDr_South	4842.0	1548.9	1.5
KestrelRidgeRd_1	4519.3	1225.7	2.7
KestrelRidgeRd_2	1221.1	371.7	1.5
KestrelRiseRd_North	1128.1	476.3	2.1
KestrelRiseRd_South	686.6	271.6	1.2
LakeRunRd_East	3297.7	1369.5	0.9
LakeRunRd_West	3619.7	2053.2	0.9
MillertonDr_North	1464.0	724.9	0.9
MillertonDr_South	1855.6	990.1	0.9
OakmondRd_East	6740.7	2593.1	2.4
OakmondRd_West	10592.5	6679.8	0.6
OpenCrestDr	9310.9	5232.1	1.8
OpenHillDr_Field	6133.4	4613.5	0.9
OquirrhLake_1	3908.3	1372.3	2.1
OquirrhLake_2	1344.0	498.7	0.9
OquirrhLake_3	3174.4	959.0	2.4
OquirrhLake_4	1565.5	634.8	1.8
OquirrhLake_5	2515.5	980.3	2.1
OquirrhLake_6	2276.2	686.2	2.1
PeekABoo_Park	1480.7	660.9	0.8
Vermillion_Park	2580.6	1656.5	0.9
Willoughby_Park	7673.7	4333.5	0.9

Rainwater Harvesting

The Rainwater Harvesting model contains 190 L rain barrels located at every home. In SWMM, discharge from rain barrels was routed to pervious surfaces in order to model the effect of capturing stormwater from rooftop runoff and then applying this water to lawns. These rain barrels treated approximately 25-30% of the impervious area of each subcatchment. The rain barrels were created and edited in the SWMM LID Editor.

Storage head drives rain barrel outflow, as such the drain coefficient can be estimated using Equation (2) by using the required time (T) to drain a depth of stored water (D, mm).

$$C = \frac{2(D^{0.5})}{T} \tag{2}$$

Using a draining time of 48 hours (USEPA 2000) in Equation (3), a C value of 0.25 for a 914.4 mm tall 190 L rain barrel was calculated. This C value was then verified with Equation (3) to validate the reasonableness of the C value. The flow through the underdrain of the rain barrel (q, mm/hr) is shown in Equation (3). In this equation C equals the drain coefficient, n is the drain exponent, h is the unit height, and H_d is the drain offset.

$$q = C\,(h - H_d)^n \tag{3}$$

Within the SWMM model LID editor, a 24 hour drain delay was included for the rain barrels. This was done to account for the lack of irrigation needed during or directly after precipitation events.

Porous Pavement

The Porous Pavement model replaces all driveways and sidewalks in the community with porous pavement. Values for input into the SWMM LID Editor were determined through the recommendations in the SWMM User's Manual (Table 27). Since each subcatchment varied with the amount of impervious cover by driveways and sidewalks, each subcatchment had a different percentage of impervious area that was treated by the porous pavement LID feature (this value ranged from 15-45% of impervious area treated).

Table 27. SWMM Parameters for PP Model

Surface Parameter	
Vegetation Volume Fraction	0
Surface Roughness (Manning's n)	0.1
Surface Slope (percent)	1.0
Pavement Parameter	
Thickness	152.4 mm
Voids Ratio (Voids/Solids)	0.2
Impervious Surface Fraction	0
Permeability	2540 mm/hr
Clogging Factor	0
Storage Parameters	
Thickness	304.8 mm
Voids Ratio (Voids/Solids)	0.4
Seepage Rate	254 mm/hr
Clogging Factor	0
Underdrain Parameters	
Flow Coefficient	0

Bioretention

This model replaced all parking strips in the community and select portions of parks with bioretention gardens. The parking strip areas were calculated within GIS. Each park was looked at individually within GIS to see if they contained any superfluous space, i.e., space that is not utilized as a playing field, playground, sitting area, etc. Typically these areas were in the corners of the parks or close to the edges. Replacing these spaces, that are without a direct beneficial use, with bioretention gardens allowed for additional LID implementation. The values of the various parameters in the SWMM LID Editor were obtained from the research conducted in the Heiberger Thesis (Heiberger 2013) and are shown in Table 28. This thesis and its resulting values were chosen due to the close proximity of the sites in the study to the Daybreak community. Each subcatchment had a different percentage of impervious area that was treated by the bioretention LID feature due to their locations (this value ranged from 20-30% of impervious area treated).

Comprehensive LID

The Comprehensive LID model incorporates all of the previously discussed LID features together in one model. Nearly all the same parameters as for each individual LID scenario were utilized. The only change was that the runoff from driveways, sidewalks, and streets was now shared with bioretention and porous pavement acting together. Thus the individual percentage of impervious area treated for each of these combinations within this model varied from the percentage of impervious area for each of these controls when they were implemented individually.

Table 28. SWMM Parameters for BR Model

Surface Parameter	
Berm Height	1253
Vegetation Volume Fraction	0.3
Surface Roughness (Manning's n)	0.15
Surface Slope (percent)	1.0
Soil Parameter	
Thickness	610 mm
Porosity (volume fraction)	0.475
Field Capacity (volume fraction)	0.378
Wilting Point (volume fraction)	0.265
Conductivity	0.6 mm/hr
Conductivity Slope	10.0
Suction Head	320 mm
Storage Parameter	
Thickness	305 mm
Voids Ratio (Voids/Solids)	0.53
Seepage Rate	34138 mm/hr
Clogging Factor	0
Underdrain Parameters	
Flow Coefficient	0

APPENDIX E

VERIFICATION

Design Storms

In order to confirm Kennecott's claim that Daybreak does in fact collect the 100-year storm event, this was modelled within SWMM using the 100-year 24-hour design storm event (Figure 17) and it was confirmed the this storm produces no outflow from the Daybreak watershed (Figure 18).

Design storms were also used in order to do the verification with the Rational Method. All design storm model simulations were run using Green-Ampt (infiltration method) and Kinematic Wave (routing method). The design storms were run from 9/01/1951 to 9/04/1951, the models was analyzed for a longer period to allow for routing of all stormwater. The reporting time step for all models is 30 minutes; runoff time step is 5 minutes for dry weather and 5 minutes for wet weather; and the routing time step is 30 seconds. The design storms used where the 2-year 24-hour, 10-year 24-hour, and the 25-year 24-hour (Figure 19).

Rational Method

Due to the lack of any data with which to calibrate the results, the reasonableness of the results was evaluated using the Rational Method in order to calculate peak flows. The Rational Method was utilized to verify the SWMM results by Equation (4). The flow rate (Q, cfs) is calculated as a product of the runoff coefficient (C, dimensionless), the rainfall intensity (I, millimeters per hour), and the drainage areas (A, hectare). NOAA Atlas 14 precipitation intensities were obtained from the NOAA website. Runoff Coefficients (C values) were determined using the Federal Highway Administration (FHWA) Introduction to Highway Hydraulics Manual and were determined through a weighted calculation to reflect the individual compositions of each of the watersheds.

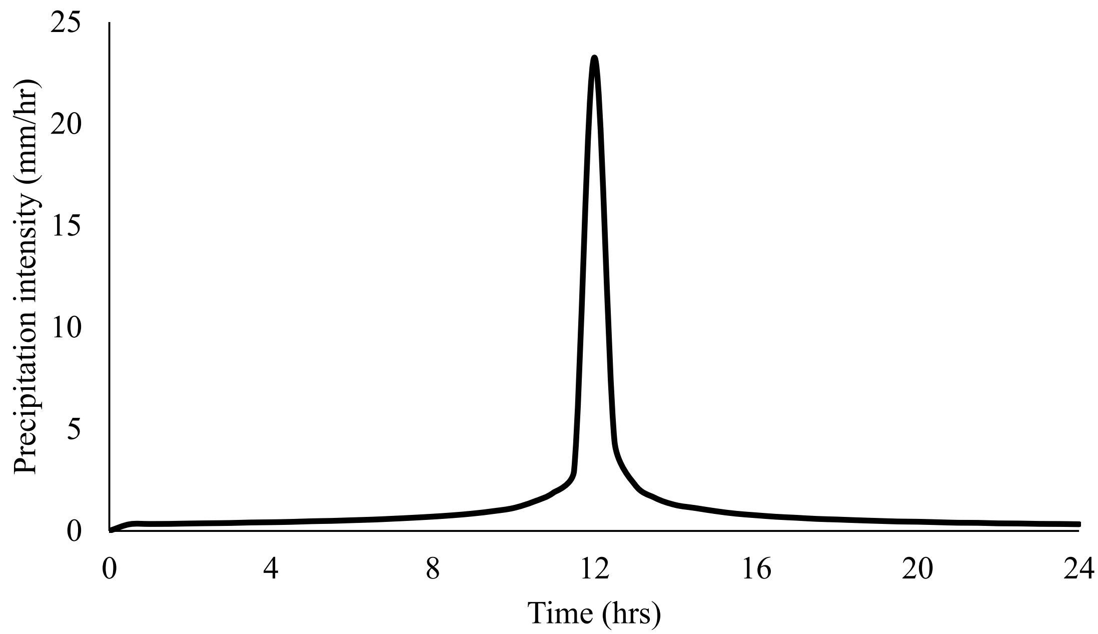

Figure 17. 100-year 24-hour Design Storm

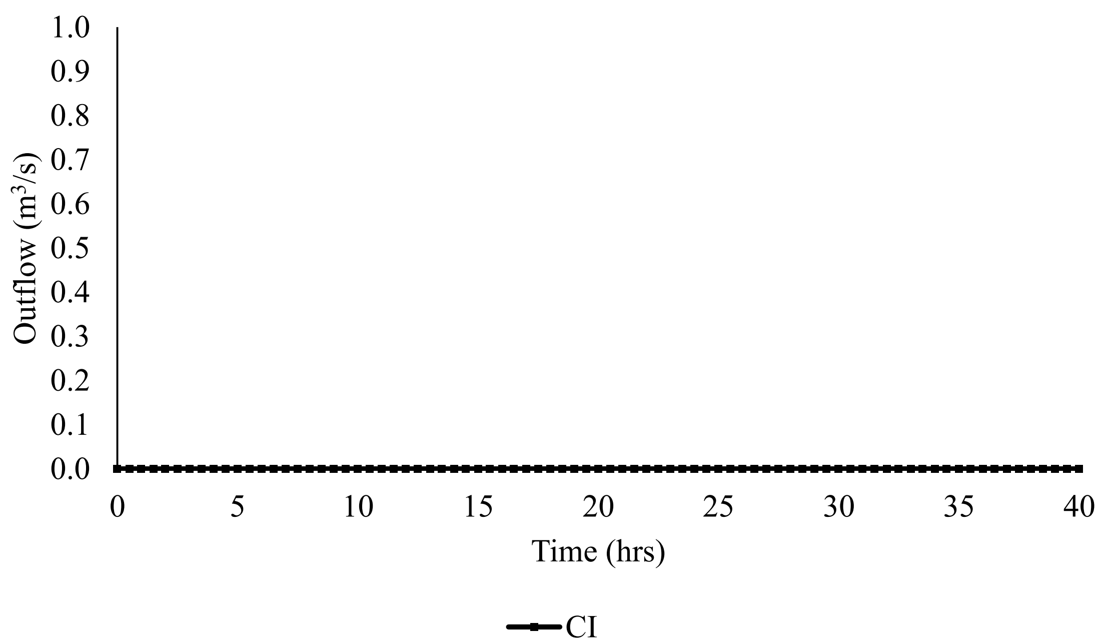

Figure 18. Outflow Results for CI Model from 100-year 24-hour Design Storm

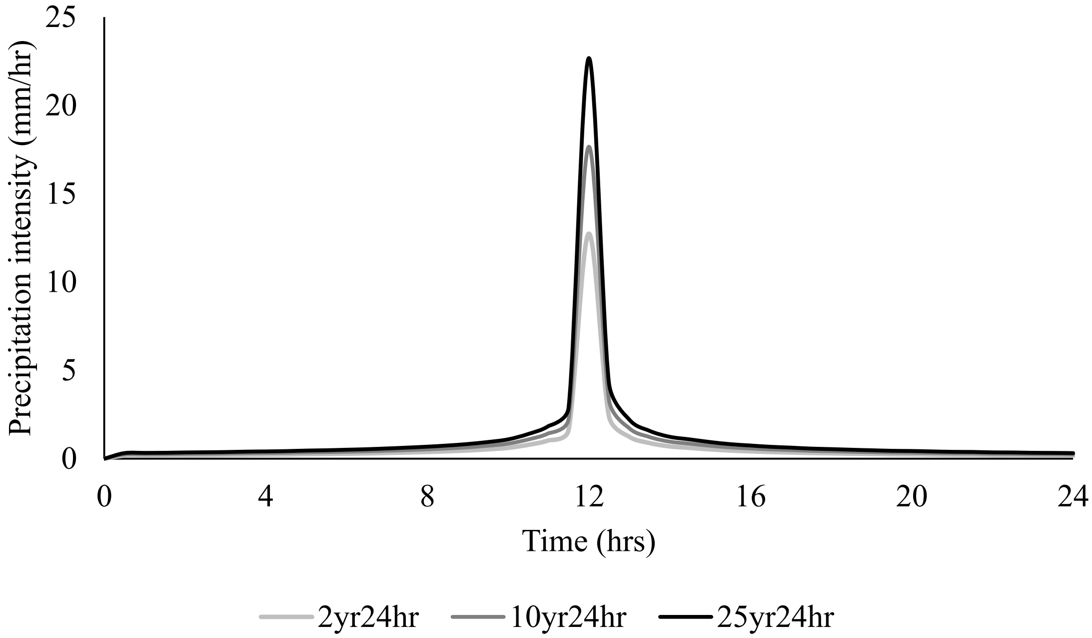

Figure 19. 2-, 10-, and 25-year 24-hour Design Storms

Since the Rational Method works best for areas below 121 hectares, runoff was verified individually for each subcatchment. This assessment was done using the NC model for the 2-, 10-, and 25-year storm events (Table 29, Table 30, and Table 31). Three representative subwatersheds were chosen in order to show the validation of the results. The models show no more than a 29% variation from the results of the rational method. The calculations in Excel are shown in Figure 20, Figure 21, and Figure 22.

$$Q = CiA \tag{4}$$

Curve Number Method

The Curve Number (CN) method was also utilized to check the validity of the runoff results of the NC model. The curve number is an empirical parameter used in

Table 29. Rational Method for FoundersParkVillage_South (70% impervious)

	FoundersParkVillage_South (70% impervious)		
Return Period	**SWMM (m^3/s)**	**Rational (m^3/s)**	**%Difference**
2 Year 24 Hour Design Storm	2.21	1.76	-23%
10 Year 24 Hour Design Storm	3.60	2.98	-19%
25 Year 24 Hour Design Storm	5.35	5.90	10%

Table 30. Rational Method for EastlakeVillage_6 (55% impervious)

	EastlakeVillage_6 (55% impervious)		
Return Period	**SWMM (m^3/s)**	**Rational (m^3/s)**	**%Difference**
2 Year 24 Hour Design Storm	0.09	0.08	-8%
10 Year 24 Hour Design Storm	0.15	0.14	-5%
25 Year 24 Hour Design Storm	0.22	0.28	25%

Table 31. Rational Method for DaybreakPkwy (40% impervious)

	DaybreakPkwy (40% impervious)		
Return Period	**SWMM (m^3/s)**	**Rational (m^3/s)**	**%Difference**
2 Year 24 Hour Design Storm	0.38	0.40	5%
10 Year 24 Hour Design Storm	0.59	0.68	15%
25 Year 24 Hour Design Storm	1.00	1.34	29%

FoundersParkVillage_South (70% impervious)

2 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	19.425	0.95
Roofs	14.569	0.85
Graded	12.141	0.45
Undeveloped	2.784	0.10
Total	48.918	**0.75**

C=	0.75	dimensionless (weighted Rational Method runoff coefficient)
I=	17.3	mm/hr
A=	48.92	hectares
Q_{PEAK}=	1.76	m^3/s, peak developed runoff

10 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	19.425	0.95
Roofs	14.569	0.85
Graded	12.141	0.45
Undeveloped	2.784	0.10
Total	48.918	**0.75**

C=	0.75	dimensionless, weighted Rational Method runoff coefficient
I=	29.2	mm/hr
A=	48.92	hectares
Q_{PEAK}=	2.98	m^3/s, peak developed runoff

25 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	19.425	0.95
Roofs	14.569	0.85
Graded	12.141	0.45
Undeveloped	2.784	0.10
Total	48.918	**0.75**

C=	0.75	dimensionless, weighted Rational Method runoff coefficient
I=	57.9	mm/hr
A=	48.92	hectares
Q_{PEAK}=	5.90	m^3/s, peak developed runoff

Figure 20. Rational Method Results for FoundersParkVillage_South Subwatershed

EastlakeVillage_6 (55% impervious)

2 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	0.891	0.95
Roofs	0.637	0.85
Graded	0.764	0.45
Undevelope	0.255	0.10
Total	2.546	**0.69**

C=	0.69	dimensionless (weighted Rational Method runoff coefficient)
I=	17.3	mm/hr
A=	2.55	hectares
Q_{PEAK}=	0.08	m^3/s, peak developed runoff

10 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	0.891	0.95
Roofs	0.637	0.85
Graded	0.764	0.45
Undevelope	0.255	0.10
Total	2.546	**0.69**

C=	0.69	dimensionless, weighted Rational Method runoff coefficient
I=	29.2	mm/hr
A=	2.55	hectares
Q_{PEAK}=	0.14	m^3/s, peak developed runoff

25 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	0.891	0.95
Roofs	0.637	0.85
Graded	0.764	0.45
Undevelope	0.255	0.10
Total	2.546	**0.69**

C=	0.69	dimensionless, weighted Rational Method runoff coefficient
I=	57.9	mm/hr
A=	2.55	hectares
Q_{PEAK}=	0.28	m^3/s, peak developed runoff

Figure 21. Rational Method Results for EastlakeVillage_6 Subwatershed

DaybreakPkwy (40% impervious)

2 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	4.952	0.95
Roofs	0.707	0.85
Graded	6.367	0.45
Undevelope	2.122	0.10
Total	14.148	**0.59**

C=	0.59	dimensionless (weighted Rational Method runoff coefficient)
I=	17.3	mm/hr
A=	14.15	hectares
Q_{PEAK}=	0.40	m^3/s, peak developed runoff

10 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	4.952	0.95
Roofs	0.707	0.85
Graded	6.367	0.45
Undevelope	2.122	0.10
Total	14.148	**0.59**

C=	0.59	dimensionless, weighted Rational Method runoff coefficient
I=	29.2	mm/hr
A=	14.15	hectares
Q_{PEAK}=	0.68	m^3/s, peak developed runoff

25 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Desc	Hectares	C
Pavement	4.952	0.95
Roofs	0.707	0.85
Graded	6.367	0.45
Undevelope	2.122	0.10
Total	14.148	**0.59**

C=	0.59	dimensionless, weighted Rational Method runoff coefficient
I=	57.9	mm/hr
A=	14.15	hectares
Q_{PEAK}=	1.34	m^3/s, peak developed runoff

Figure 22. Rational Method Results for DaybreakPkwy Subwatershed

hydrology for predicting direct runoff (this method was developed by the USDA Natural Resources Conservation Service). It may be used to determine an approximate amount of runoff from a storm event. In this thesis, the design storms were used to compare calculations done in Excel to those done in SWMM. A curve number is based on an area's hydrologic soil group, land use, treatment and hydrologic condition using the SCS Runoff Equation (Equation 5 and 6).

Runoff was verified for the NC models for the 2, 10, and 25-year storm events (Table 32, Table 33, and Table 34). Three storms from the long-term record were chosen and several representative subwatersheds were chosen in order to validate the results. The models show no more than a 31% variation from the results of the rational method. The calculations in Excel are shown in Figure 23, Figure 24, and Figure 25.

$$Q = \frac{(P-0.2S)^2}{(P+0.8S)} \tag{5}$$

where:

Q= runoff (mm)

P= rainfall (mm)

S= potential maximum retention after runoff begins (mm)

$$S = \frac{1000}{CN} - 10 \tag{6}$$

where:

S= potential maximum retention after runoff begins (mm)

CN = dimensionless Curve Number

Table 32. CN Method for FoundersParkVillage_South (70% impervious)

	FoundersParkVillage_South (70% impervious)		
Return Period	**SWMM (m^3)**	**CN (m^3)**	**%Difference**
2 Year 24 Hour Design Storm	4496	4254	-6%
10 Year 24 Hour Design Storm	6991	8362	18%
25 Year 24 Hour Design Storm	9633	13173	31%

Table 33. CN Method for EastlakeVillage_6 (55% impervious)

	EastlakeVillage_6 (55% impervious)		
Return Period	**SWMM (m^3)**	**CN (m^3)**	**%Difference**
2 Year 24 Hour Design Storm	167	128	-26%
10 Year 24 Hour Design Storm	270	303	11%
25 Year 24 Hour Design Storm	395	525	28%

Table 34. CN Method for DaybreakPkwy (40% impervious)

	DaybreakPkwy (40% impervious)		
Return Period	**SWMM (m^3)**	**CN (m^3)**	**%Difference**
2 Year 24 Hour Design Storm	404	331	-20%
10 Year 24 Hour Design Storm	1054	984	-7%
25 Year 24 Hour Design Storm	1442	1890	27%

FoundersParkVillage_South (70% impervious)

2 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	19.425	94
Roofs	14.569	98
Graded	12.141	61
Undeveloped	2.784	60
Total	48.918	**85**

CN =	85	dimensionless, weighted Curve Number runoff coefficient
S =	45	mm
P =	34	mm
Q =	9	mm
	4254	m^3

10 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	19.425	94
Roofs	14.569	98
Graded	12.141	61
Undeveloped	2.784	60
Total	48.918	**85**

CN =	85	dimensionless, weighted Curve Number runoff coefficient
S =	45	mm
P =	46	mm
Q =	17	mm
	8362	m^3

25 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	19.425	94
Roofs	14.569	98
Graded	12.141	61
Undeveloped	2.784	60
Total	48.918	**85**

CN =	85	dimensionless, weighted Curve Number runoff coefficient
S =	45	mm
P =	60	mm
Q =	27	mm
	13173	m^3

Figure 23. CN Method Results for FoundersParkVillage_South Subwatershed

EastlakeVillage_6 (55% impervious)

2 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	0.891	87
Roofs	0.637	98
Graded	0.764	62
Undeveloped	0.255	56
Total	2.546	**79**

CN =	79	dimensionless, weighted Curve Number runoff coefficient
S =	68	mm
P =	34	mm
Q =	5	mm
	128	m^3

10 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	0.891	87
Roofs	0.637	98
Graded	0.764	62
Undeveloped	0.255	56
Total	2.546	**79**

CN =	79	dimensionless, weighted Curve Number runoff coefficient
S =	68	mm
P =	46	mm
Q =	11	mm
	303	m^3

25 Year 24 Hour Design Storm

weighted CN Method runoff coefficient

Description	Hectares	C
Pavement	0.891	87
Roofs	0.637	98
Graded	0.764	62
Undeveloped	0.255	56
Total	2.546	**79**

CN =	79	dimensionless, weighted Curve Number runoff coefficient
S =	68	mm
P =	60	mm
Q =	19	mm
	525	m^3

Figure 24. CN Method Results for EastlakeVillage_6 Subwatershed

DaybreakPkwy (40% impervious)

2 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	4.952	93
Roofs	0.707	98
Graded	6.367	63
Undeveloped	2.122	56
Total	14.148	**74**

CN =	74	dimensionless, weighted Curve Number runoff coefficient
S =	89	mm
P =	34	mm
Q =	2	mm
	331	m^3

10 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	4.952	93
Roofs	0.707	98
Graded	6.367	63
Undeveloped	2.122	56
Total	14.148	**74**

CN =	74	dimensionless, weighted Curve Number runoff coefficient
S =	89	mm
P =	46	mm
Q =	7	mm
	984	m^3

25 Year 24 Hour Design Storm

weighted Rational Method runoff coefficient

Description	Hectares	C
Pavement	4.952	93
Roofs	0.707	98
Graded	6.367	63
Undeveloped	2.122	56
Total	14.148	**74**

CN =	74	dimensionless, weighted Curve Number runoff coefficient
S =	89	mm
P =	60	mm
Q =	13	mm
	1890	m^3

Figure 25. CN Method Results for DaybreakPkwy Subwatershed

APPENDIX F

STATISTICAL SIGNIFICANCE

% Reduction for All Years

F-Test Two-Sample for Variances

% REDUCTION - TOTAL VOLUMES	*Centralized Infiltration*	*Comprehensive LID*
Mean	0.9946	0.9671
Variance	0.0001	0.0000
Observations	60	60
df	59	59
F	281.36	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and $p<0.05$ (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - TOTAL VOLUMES	*Centralized Infiltration*	*Comprehensive LID*
Mean	0.9945513	0.9671332
Variance	0.0001119	0.0000004
Observations	60	60
Hypothesized Mean Difference	0	
df	59	
t Stat	20.04	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.00	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

% REDUCTION - TOTAL VOLUMES	*Porous Pavement*	*Bioretention*
Mean	0.673724	0.624818
Variance	0.000009	0.000002
Observations	60	60
df	59	59
F	4.28	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - TOTAL VOLUMES	*Porous Pavement*	*Bioretention*
Mean	0.673724	0.624818
Variance	0.000009	0.000002
Observations	60	60
Hypothesized Mean Difference	0	
df	85	
t Stat	114.19	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.99	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

% REDUCTION - MEAN FLOW	*Porous Pavement*	*Bioretention*
Mean	0.59014	0.57987
Variance	0.00009	0.00003
Observations	60	60
df	59	59
F	2.66	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - MEAN FLOW	*Porous Pavement*	*Bioretention*
Mean	0.59014	0.57987
Variance	0.00009	0.00003
Observations	60	60
Hypothesized Mean Difference	0	
df	98	
t Stat	7.31	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for
Variances

% REDUCTION - MEAN FLOW	*Centralized Infiltration*	*Porous Pavement*
Mean	0.64932	0.59014
Variance	0.95063	0.00009
Observations	60	60
df	59	59
F	11028.90	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - MEAN FLOW	*Centralized Infiltration*	*Porous Pavement*
Mean	0.6493	0.5901
Variance	0.9506	0.0001
Observations	60	60
Hypothesized Mean Difference	0	
df	59	
t Stat	0.47	
P(T<=t) one-tail	0.32	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.64	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

% REDUCTION - PEAK FLOW	*Centralized Infiltration*	*Comprehensive LID*
Mean	0.9643	0.9635
Variance	0.0036	0.0000
Observations	60	60
df	59	59
F	295.14	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - PEAK FLOW	*Centralized Infiltration*	*Comprehensive LID*
Mean	0.96428	0.96350
Variance	0.00356	0.00001
Observations	60	60
Hypothesized Mean Difference	0	
df	59	
t Stat	0.10	
P(T<=t) one-tail	0.46	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.92	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

% REDUCTION - PEAK FLOW	*Porous Pavement*	*Bioretention*
Mean	0.6602	0.6220
Variance	0.0003	0.0001
Observations	60	60
df	59	59
F	2.84	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - PEAK FLOW	*Porous Pavement*	*Bioretention*
Mean	0.6602	0.6220
Variance	0.0003	0.0001
Observations	60	60
Hypothesized Mean Difference	0	
df	96	
t Stat	14.26	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

Average Total Volume

F-Test Two-Sample for Variances

AVERAGE VOLUME	*No Controls*	*Rainwater Harvesting*
Mean	689.3487274	453.2786075
Variance	34384.2082	15043.89424
Observations	60	60
df	59	59
F	2.29	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

AVERAGE VOLUME	*No Controls*	*Rainwater Harvesting*
Mean	689.3487274	453.2786075
Variance	34384.2082	15043.89424
Observations	60	60
Hypothesized Mean Difference	0	
df	102	
t Stat	8.22	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

AVERAGE VOLUME	*Centralized Infiltration*	*Predevelopment*
Mean	4.38329847	9.863489177
Variance	81.80791777	22.37277962
Observations	60	60
df	59	59
F	3.66	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

AVERAGE VOLUME	*Centralized Infiltration*	*Predevelopment*
Mean	4.38329847	9.863489177
Variance	81.80791777	22.37277962
Observations	60	60
Hypothesized Mean Difference	0	
df	89	
t Stat	-4.16	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.99	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

AVERAGE VOLUME	*Comprehensive LID*	*Predevelopment*
Mean	22.63210292	9.863489177
Variance	35.94526057	22.37277962
Observations	60	60
df	59	59
F	1.61	
P(F<=f) one-tail	0.04	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

AVERAGE VOLUME	*Comprehensive LID*	*Predevelopment*
Mean	22.63210292	9.863489177
Variance	35.94526057	22.37277962
Observations	60	60
Hypothesized Mean Difference	0	
df	112	
t Stat	12.95	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

AVERAGE VOLUME	*Rainwater Harvesting*	*Bioretention*
Mean	453.28	258.57
Variance	15043.89	4814.30
Observations	60	60
df	59	59
F	3.12	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

AVERAGE VOLUME	*Rainwater Harvesting*	*Bioretention*
Mean	453.28	258.57
Variance	15043.89	4814.30
Observations	60	60
Hypothesized Mean Difference	0	
df	93	
t Stat	10.70	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.99	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

AVERAGE VOLUME	*Bioretention*	*Porous Pavement*
Mean	258.57	224.79
Variance	4814.30	3597.42
Observations	60	60
df	59	59
F	1.34	
P(F<=f) one-tail	0.13	
F Critical one-tail	1.54	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

AVERAGE VOLUME	*Bioretention*	*Porous Pavement*
Mean	258.57	224.79
Variance	4814.30	3597.42
Observations	60	60
Pooled Variance	4205.86	
Hypothesized Mean Difference	0	
df	118	
t Stat	2.85	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.01	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

AVERAGE VOLUME	*Porous Pavement*	*Comprehensive LID*
Mean	224.79	22.63
Variance	3597.42	35.95
Observations	60	60
df	59	59
F	100.08	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

AVERAGE VOLUME	*Porous Pavement*	*Comprehensive LID*
Mean	224.79	22.63
Variance	3597.42	35.95
Observations	60	60
Hypothesized Mean Difference	0	
df	60	
t Stat	25.98	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.00	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

Volume – Wet/Dry Years

F-Test Two-Sample for Variances

	Centralized Infiltration	
VOLUME TOTALS	*Wet*	*Dry*
Mean	8.514624451	1.151941498
Variance	138.3100726	6.672162085
Observations	20	19
df	19	18
F	20.73	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Centralized Infiltration	
VOLUME TOTALS	*Wet*	*Dry*
Mean	8.514624451	1.151941498
Variance	138.3100726	6.672162085
Observations	20	19
Hypothesized Mean Difference	0	
df	21	
t Stat	2.73	
P(T<=t) one-tail	0.01	
t Critical one-tail	1.72	
P(T<=t) two-tail	0.01	
t Critical two-tail	2.08	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

VOLUME TOTALS	Rainwater Harvesting *Wet*	Rainwater Harvesting *Dry*
Mean	571.1049335	321.69246
Variance	10362.98793	2537.209762
Observations	20	19
df	19	18
F	4.08	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

VOLUME TOTALS	Rainwater Harvesting *Wet*	Rainwater Harvesting *Dry*
Mean	571.1049335	321.69246
Variance	10362.98793	2537.209762
Observations	20	19
Hypothesized Mean Difference	0	
df	28	
t Stat	9.77	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

VOLUME TOTALS	Bioretention Wet	Bioretention Dry
Mean	325.3050514	184.3719489
Variance	3393.847635	788.1050457
Observations	20	19
df	19	18
F	4.31	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

VOLUME TOTALS	Bioretention Wet	Bioretention Dry
Mean	325.3050514	184.3719489
Variance	3393.847635	788.1050457
Observations	20	19
Hypothesized Mean Difference	0	
df	28	
t Stat	9.70	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Porous Pavement	
VOLUME TOTALS	*Wet*	*Dry*
Mean	282.055185	160.436138
Variance	2545.034529	587.4466524
Observations	20	19
df	19	18
F	4.33	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Porous Pavement	
VOLUME TOTALS	*Wet*	*Dry*
Mean	282.055185	160.436138
Variance	2545.034529	587.4466524
Observations	20	19
Hypothesized Mean Difference	0	
df	28	
t Stat	9.67	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Comprehensive LID	
VOLUME TOTALS	*Wet*	*Dry*
Mean	28.29385235	16.1943503
Variance	25.81929506	6.108498167
Observations	20	19
df	19	18
F	4.23	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Comprehensive LID	
VOLUME TOTALS	*Wet*	*Dry*
Mean	28.29385235	16.1943503
Variance	25.81929506	6.108498167
Observations	20	19
Hypothesized Mean Difference	0	
df	28	
t Stat	9.53	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

Volume % Reduction for Wet/Dry Years

F-Test Two-Sample for Variances

	Storage	
% REDUCTION - AVERAGE VOLUMES	*Wet*	*Dry*
Mean	0.99073	0.99780
Variance	0.00014	0.00002
Observations	20	19
df	19	18
F	6.16	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Storage	
% REDUCTION - AVERAGE VOLUMES	*Wet*	*Dry*
Mean	0.99073	0.99780
Variance	0.00014	0.00002
Observations	20	19
Hypothesized Mean Difference	0	
df	25	
t Stat	-2.44	
P(T<=t) one-tail	0.01	
t Critical one-tail	1.71	
P(T<=t) two-tail	0.02	
t Critical two-tail	2.06	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

% REDUCTION - AVERAGE VOLUMES	Rainwater Harvesting Dry	Wet
Mean	0.344632	0.342129
Variance	0.000030	0.000015
Observations	19	20
df	18	19
F	2.01	
P(F<=f) one-tail	0.07	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

% REDUCTION - AVERAGE VOLUMES	Rainwater Harvesting Dry	Wet
Mean	0.344632	0.342129
Variance	0.000030	0.000015
Observations	19	20
Pooled Variance	2E-05	
Hypothesized Mean Difference	0	
df	37	
t Stat	1.65	
P(T<=t) one-tail	0.05	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.11	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

	Bioretention	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.624126	0.625399
Variance	0.000002	0.000001
Observations	19	20
df	18	19
F	1.28	
P(F<=f) one-tail	0.30	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Bioretention	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.624126	0.625399
Variance	0.000002	0.000001
Observations	19	20
Pooled Variance	1.64E-06	
Hypothesized Mean Difference	0	
df	37	
t Stat	-3.10	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.03	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Porous Pavement	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.672850	0.675238
Variance	0.000004	0.000003
Observations	19	20
df	18	19
F	1.51	
P(F<=f) one-tail	0.19	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Porous Pavement	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.672850	0.675238
Variance	0.000004	0.000003
Observations	19	20
Pooled Variance	3.17E-06	
Hypothesized Mean Difference	0	
df	37	
t Stat	-4.18	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.03	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Comprehensive LID	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.9669794	0.9674350
Variance	0.0000003	0.0000001
Observations	19	20
df	18	19
F	3.77	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Comprehensive LID	
% REDUCTION - AVERAGE VOLUMES	*Dry*	*Wet*
Mean	0.9669794	0.9674350
Variance	0.0000003	0.0000001
Observations	19	20
Hypothesized Mean Difference	0	
df	27	
t Stat	-3.13	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

Average Mean Flows

F-Test Two-Sample for Variances

MEAN FLOW	*Predevelopment*	*Comprehensive LID*
Mean	0.01341	0.01160
Variance	0.00006	0.00000
Observations	60	60
df	59	59
F	26.20	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	*Predevelopment*	*Comprehensive LID*
Mean	0.01341	0.01160
Variance	0.00006	0.00000
Observations	60	60
Hypothesized Mean Difference	0	
df	63	
t Stat	1.83	
P(T<=t) one-tail	0.04	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.07	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

MEAN FLOW	*No Controls*	*Rainwater Harvesting*
Mean	0.1796	0.1235
Variance	0.0006	0.0003
Observations	60	60
df	59	59
F	2.04	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	*No Controls*	*Rainwater Harvesting*
Mean	0.1796	0.1235
Variance	0.0006	0.0003
Observations	60	60
Hypothesized Mean Difference	0	
df	106	
t Stat	14.24	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

MEAN FLOW	*Centralized Infiltration*	*Bioretention*
Mean	0.0705	0.0754
Variance	0.0428	0.0001
Observations	60	60
df	59	59
F	436.88	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	*Centralized Infiltration*	*Bioretention*
Mean	0.0705	0.0754
Variance	0.0428	0.0001
Observations	60	60
Hypothesized Mean Difference	0	
df	59	
t Stat	-0.18	
P(T<=t) one-tail	0.43	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.86	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

MEAN FLOW	*Centralized Infiltration*	*Porous Pavement*
Mean	0.070	0.073
Variance	0.043	0.000
Observations	60	60
df	59	59
F	457.26	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	*Centralized Infiltration*	*Porous Pavement*
Mean	0.070	0.073
Variance	0.043	0.000
Observations	60	60
Hypothesized Mean Difference	0	
df	59	
t Stat	-0.11	
P(T<=t) one-tail	0.46	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.91	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

MEAN FLOW	*Rainwater Harvesting*	*Bioretention*
Mean	0.1235	0.0754
Variance	0.0003	0.0001
Observations	60	60
df	59	59
F	3.12	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	*Rainwater Harvesting*	*Bioretention*
Mean	0.1235	0.0754
Variance	0.0003	0.0001
Observations	60	60
Hypothesized Mean Difference	0	
df	93	
t Stat	18.55	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.99	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

MEAN FLOW	*Bioretention*	*Porous Pavement*
Mean	0.07535	0.07350
Variance	0.00010	0.00009
Observations	60	60
df	59	59
F	1.05	
P(F<=f) one-tail	0.43	
F Critical one-tail	1.54	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

MEAN FLOW	*Bioretention*	*Porous Pavement*
Mean	0.07535	0.07350
Variance	0.00010	0.00009
Observations	60	60
Pooled Variance	1E-04	
Hypothesized Mean Difference	0	
df	118	
t Stat	1.04	
P(T<=t) one-tail	0.15	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.30	
t Critical two-tail	1.98	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

Mean Flows – Wet/Dry Years

F-Test Two-Sample for Variances

	Centralized Infiltration	
MEAN FLOW	*Wet*	*Dry*
Mean	0.091	0.026
Variance	0.009	0.003
Observations	20	19
df	19	18
F	3.35	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Centralized Infiltration	
MEAN FLOW	*Wet*	*Dry*
Mean	0.091	0.026
Variance	0.009	0.003
Observations	20	19
Hypothesized Mean Difference	0	
df	30	
t Stat	2.63	
P(T<=t) one-tail	0.01	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.01	
t Critical two-tail	2.04	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Rainwater Harvesting	
MEAN FLOW	*Dry*	*Wet*
Mean	0.1087	0.1348
Variance	0.0003	0.0001
Observations	19	20
df	18	19
F	3.22	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Rainwater Harvesting	
MEAN FLOW	*Dry*	*Wet*
Mean	0.1087	0.1348
Variance	0.0003	0.0001
Observations	19	20
Hypothesized Mean Difference	0	
df	28	
t Stat	-5.68	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

MEAN FLOW	Bioretention Dry	Bioretention Wet
Mean	0.06700	0.08176
Variance	0.00010	0.00004
Observations	19	20
df	18	19
F	2.83	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

MEAN FLOW	Bioretention Dry	Bioretention Wet
Mean	0.06700	0.08176
Variance	0.00010	0.00004
Observations	19	20
Hypothesized Mean Difference	0	
df	29	
t Stat	-5.51	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Porous Pavement	
MEAN FLOW	*Dry*	*Wet*
Mean	0.06565	0.07934
Variance	0.00010	0.00003
Observations	19	20
df	18	19
F	3.03	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Porous Pavement	
MEAN FLOW	*Dry*	*Wet*
Mean	0.06565	0.07934
Variance	0.00010	0.00003
Observations	19	20
Hypothesized Mean Difference	0	
df	28	
t Stat	-5.16	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Comprehensive LID	
MEAN FLOW	*Dry*	*Wet*
Mean	0.010647	0.012241
Variance	0.000003	0.000001
Observations	19	20
df	18	19
F	4.28	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Comprehensive LID	
MEAN FLOW	*Dry*	*Wet*
Mean	0.010647	0.012241
Variance	0.000003	0.000001
Observations	19	20
Hypothesized Mean Difference	0	
df	26	
t Stat	-3.56	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.71	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.06	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

Mean Flows % Reduction for Wet/Dry Years

F-Test Two-Sample for Variances

	Storage	
% REDUCTION - AVERAGE MEAN FLOWS	*Wet*	*Dry*
Mean	0.55	0.86
Variance	0.23	0.08
Observations	20	19
df	19	18
F	2.80	
P(F<=f) one-tail	0.02	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Storage	
% REDUCTION - AVERAGE MEAN FLOWS	*Wet*	*Dry*
Mean	0.546480745	0.856140471
Variance	0.232331772	0.083042713
Observations	20	19
Hypothesized Mean Difference	0	
df	31	
t Stat	-2.45	
P(T<=t) one-tail	0.01	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.02	
t Critical two-tail	2.04	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Rainwater Harvesting	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.31148	0.31357
Variance	0.00004	0.00002
Observations	19	20
df	18	19
F	1.72	
P(F<=f) one-tail	0.13	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Rainwater Harvesting	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.31148266	0.313572585
Variance	4.21233E-05	2.45129E-05
Observations	19	20
Pooled Variance	3.30801E-05	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.13	
P(T<=t) one-tail	0.13	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.26	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

	Bioretention	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.57502	0.58361
Variance	0.00004	0.00001
Observations	19	20
df	18	19
F	5.15	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Bioretention	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.57502	0.58361
Variance	0.00004	0.00001
Observations	19	20
Hypothesized Mean Difference	0	
df	24	
t Stat	-5.47	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.71	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.06	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Porous Pavement	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.58367	0.59591
Variance	0.00009	0.00002
Observations	19	20
df	18	19
F	3.90	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Porous Pavement	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.583666	0.595910
Variance	0.000095	0.000024
Observations	19	20
Hypothesized Mean Difference	0	
df	26	
t Stat	-4.91	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.71	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.06	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Comprehensive LID	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.932511	0.937614
Variance	0.000013	0.000004
Observations	19	20
df	18	19
F	3.41	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.18	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

	Comprehensive LID	
% REDUCTION - AVERAGE MEAN FLOWS	*Dry*	*Wet*
Mean	0.932511	0.937614
Variance	0.000013	0.000004
Observations	19	20
Hypothesized Mean Difference	0	
df	27	
t Stat	-5.39	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.00	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

Average Peaks

F-Test Two-Sample for Variances

PEAK FLOW	*No Controls*	*Rainwater Harvesting*
Mean	6.572	4.571
Variance	28.805	17.430
Observations	60	60
df	59	59
F	1.65	
P(F<=f) one-tail	0.03	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

PEAK FLOW	*No Controls*	*Rainwater Harvesting*
Mean	6.572	4.571
Variance	28.805	17.430
Observations	60	60
Hypothesized Mean Difference	0	
df	111	
t Stat	2.28	
P(T<=t) one-tail	0.01	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.02	
t Critical two-tail	1.98	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

PEAK FLOW	*Rainwater Harvesting*	*Bioretention*
Mean	4.571	2.464
Variance	17.430	3.772
Observations	60	60
df	59	59
F	4.62	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

PEAK FLOW	*Rainwater Harvesting*	*Bioretention*
Mean	4.571	2.464
Variance	17.430	3.772
Observations	60	60
Hypothesized Mean Difference	0	
df	83	
t Stat	3.54	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.00	
t Critical two-tail	1.99	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

PEAK FLOW	*Porous Pavement*	*Bioretention*
Mean	2.249	2.464
Variance	4.101	3.772
Observations	60	60
df	59	59
F	1.09	
P(F<=f) one-tail	0.37	
F Critical one-tail	1.54	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

PEAK FLOW	*Porous Pavement*	*Bioretention*
Mean	2.249	2.464
Variance	4.101	3.772
Observations	60	60
Pooled Variance	3.937	
Hypothesized Mean Difference	0	
df	118	
t Stat	-0.59	
P(T<=t) one-tail	0.28	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.56	
t Critical two-tail	1.98	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

PEAK FLOW	*Predevelopment*	*Comprehensive LID*
Mean	0.25	0.25
Variance	1.14	0.06
Observations	60	60
df	59	59
F	18.09	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

PEAK FLOW	*Predevelopment*	*Comprehensive LID*
Mean	0.246	0.250
Variance	1.136	0.063
Observations	60	60
Hypothesized Mean Difference	0	
df	66	
t Stat	-0.03	
P(T<=t) one-tail	0.49	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.98	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

PEAK FLOW	*Predevelopment*	*Centralized Infiltration*
Mean	0.246	0.315
Variance	1.136	0.940
Observations	60	60
df	59	59
F	1.21	
P(F<=f) one-tail	0.23	
F Critical one-tail	1.54	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

PEAK FLOW	*Predevelopment*	*Centralized LID*
Mean	0.246	0.315
Variance	1.136	0.940
Observations	60	60
Pooled Variance	1.038	
Hypothesized Mean Difference	0	
df	118	
t Stat	-0.37	
P(T<=t) one-tail	0.36	
t Critical one-tail	1.66	
P(T<=t) two-tail	0.71	
t Critical two-tail	1.98	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

PEAK FLOW	*Centralized Infiltration*	*Comprehensive LID*
Mean	0.315	0.250
Variance	0.940	0.063
Observations	60	60
df	59	59
F	14.97	
P(F<=f) one-tail	0.00	
F Critical one-tail	1.54	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

PEAK FLOW	*Centralized LID*	*Comprehensive LID*
Mean	0.315	0.250
Variance	0.940	0.063
Observations	60	60
Hypothesized Mean Difference	0	
df	67	
t Stat	0.50	
P(T<=t) one-tail	0.31	
t Critical one-tail	1.67	
P(T<=t) two-tail	0.62	
t Critical two-tail	2.00	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

Peaks – Wet/Dry Years

F-Test Two-Sample for Variances

	Centralized Infiltration	
PEAK FLOW	*Wet*	*Dry*
Mean	0.433	0.102
Variance	0.219	0.044
Observations	20	19
df	19	18
F	4.95	
P(F<=f) one-tail	0.00	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05
(Variances are unequal)

t-Test: Two-Sample Assuming Unequal
Variances

	Centralized Infiltration	
PEAK FLOW	*Wet*	*Dry*
Mean	0.433	0.102
Variance	0.219	0.044
Observations	20	19
Hypothesized Mean Difference	0	
df	27	
t Stat	2.87	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.01	
t Critical two-tail	2.05	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat > t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

PEAK FLOW	Rainwater Harvesting Dry	Rainwater Harvesting Wet
Mean	3.53	4.75
Variance	4.45	3.99
Observations	19	20
df	18	19
F	1.11	
P(F<=f) one-tail	0.41	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

PEAK FLOW	Rainwater Harvesting Dry	Rainwater Harvesting Wet
Mean	3.53	4.75
Variance	4.45	3.99
Observations	19	20
Pooled Variance	4.21	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.86	
P(T<=t) one-tail	0.04	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.07	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

	Bioretention	
PEAK FLOW	*Dry*	*Wet*
Mean	1.97	2.60
Variance	1.17	1.14
Observations	19	20
df	18	19
F	1.03	
P(F<=f) one-tail	0.47	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05
(Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Bioretention	
PEAK FLOW	*Dry*	*Wet*
Mean	1.966	2.602
Variance	1.173	1.138
Observations	19	20
Pooled Variance	1.155	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.85	
P(T<=t) one-tail	0.04	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.07	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

PEAK FLOW	Porous Pavement Wet	Porous Pavement Dry
Mean	2.33	1.76
Variance	0.93	0.92
Observations	20	19
df	19	18
F	1.01	
P(F<=f) one-tail	0.49	
F Critical one-tail	2.20	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05
(Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

PEAK FLOW	Porous Pavement Wet	Porous Pavement Dry
Mean	2.33	1.76
Variance	0.93	0.92
Observations	20	19
Pooled Variance	0.92	
Hypothesized Mean Difference	0	
df	37	
t Stat	1.83	
P(T<=t) one-tail	0.04	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.08	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

	Comprehensive LID	
PEAK FLOW	*Dry*	*Wet*
Mean	0.192	0.253
Variance	0.016	0.014
Observations	19	20
df	18	19
F	1.08	
P(F<=f) one-tail	0.43	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Comprehensive LID	
PEAK FLOW	*Dry*	*Wet*
Mean	0.192	0.253
Variance	0.016	0.014
Observations	19	20
Pooled Variance	0.015	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.56	
P(T<=t) one-tail	0.06	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.13	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

Peaks % Reduction

F-Test Two-Sample for Variances

	Storage	
% REDUCTION - AVERAGE PEAK FLOWS	*Wet*	*Dry*
Mean	0.9305	0.9826
Variance	0.0051	0.0017
Observations	20	19
df	19	18
F	3.08	
P(F<=f) one-tail	0.01	
F Critical one-tail	2.20	

Reject the null hypothesis b/c F > F Critical and p<0.05 (Variances are unequal)

t-Test: Two-Sample Assuming Unequal Variances

% REDUCTION - AVERAGE PEAK FLOWS	*Wet*	*Dry*
Mean	0.9305	0.9826
Variance	0.0051	0.0017
Observations	20	19
Hypothesized Mean Difference	0	
df	30	
t Stat	-2.82	
P(T<=t) one-tail	0.00	
t Critical one-tail	1.70	
P(T<=t) two-tail	0.01	
t Critical two-tail	2.04	

Reject the null hypothesis b/c P two-tail < alpha (0.05)
Reject the null hypothesis b/c t Stat < -t Critical two tail

DIFFERENT

F-Test Two-Sample for Variances

	Rainwater Harvesting	
% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.3275	0.3136
Variance	0.0006	0.0004
Observations	19	20
df	18	19
F	1.35	
P(F<=f) one-tail	0.26	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.327532252	0.313591855
Variance	0.00056804	0.000419312
Observations	19	20
Pooled Variance	0.000491666	
Hypothesized Mean Difference	0	
df	37	
t Stat	1.96	
P(T<=t) one-tail	0.03	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.06	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat < t Critical two tail

SAME

F-Test Two-Sample for Variances

	Bioretention	
% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.61845	0.62403
Variance	0.00014	0.00009
Observations	19	20
df	18	19
F	1.50	
P(F<=f) one-tail	0.19	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Bioretention	
% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.61845	0.62403
Variance	0.00014	0.00009
Observations	19	20
Pooled Variance	0.00011757	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.61	
P(T<=t) one-tail	0.06	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.12	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

	Porous Pavement	
% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.65578	0.66468
Variance	0.00043	0.00022
Observations	19	20
df	18	19
F	1.92	
P(F<=f) one-tail	0.08	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

	Porous Pavement	
% REDUCTION - AVERAGE PEAK FLOWS	*Dry*	*Wet*
Mean	0.6558	0.6647
Variance	0.0004	0.0002
Observations	19	20
Pooled Variance	0.000321971	
Hypothesized Mean Difference	0	
df	37	
t Stat	-1.55	
P(T<=t) one-tail	0.07	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.13	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

F-Test Two-Sample for Variances

% REDUCTION - AVERAGE PEAK FLOWS	Comprehensive LID *Dry*	*Wet*
Mean	0.963166	0.964155
Variance	0.000013	0.000008
Observations	19	20
df	18	19
F	1.64	
P(F<=f) one-tail	0.15	
F Critical one-tail	2.18	

Cannot reject the null hypothesis b/c F < F Critical and p>0.05 (Variances are equal)

t-Test: Two-Sample Assuming Equal Variances

% REDUCTION - AVERAGE PEAK FLOWS	Comprehensive LID *Dry*	*Wet*
Mean	0.963166	0.964155
Variance	0.000013	0.000008
Observations	19	20
Pooled Variance	1.06277E-05	
Hypothesized Mean Difference	0	
df	37	
t Stat	-0.95	
P(T<=t) one-tail	0.18	
t Critical one-tail	1.69	
P(T<=t) two-tail	0.35	
t Critical two-tail	2.03	

Cannot reject the null hypothesis b/c P two-tail > alpha (0.05)
Cannot reject the null hypothesis b/c t Stat > -t Critical two tail

SAME

LITERATURE CITED

ASCE, 1992. Design & Construction of Urban Stormwater Management Systems, New York, NY.

Barco J., Wong K.M., & Stenstrom M.K., 2008. Automatic calibration of the US EPA SWMM model for a large urban catchment. J Hydraul Eng;134:466–74.

Bedan, E. S., & Clausen, J. C., 2009. Stormwater runoff quality and quantity from traditional and low impact development watersheds1. JAWRA Journal of the American Water Resources Association, 45(4), 998-1008.

Berke, P. R., MacDonald, J., White, N., Holmes, M., Line, D., Oury, K., & Ryznar, R. (2003). Greening development to protect watersheds: does new urbanism make a difference? Journal of the American Planning Association, 69(4), 397-413.

Bierwagen, B. G., Theobald, D. M., Pyke, C. R., Choate, A., Groth, P., Thomas, J. V., & Morefield, P., 2010. National housing and impervious surface scenarios for integrated climate impact assessments. Proceedings of the National Academy of Sciences, 107(49), 20887-20892.

Brander, K. E., Owen, K. E., & Potter, K. W., 2004. Modeled Impacts of Development Type on Runoff Volume and Infiltration Performance. JAWRA Journal of the American Water Resources Association, 40(4), 961-969.

Burns M., Fletcher T.D., Hatt B.E., Ladson A., & Walsh C.J., 2012. Hydrologic shortcomings of conventional urban stormwater management and opportunities for reform. Landscape Urban Plan;105:230–40.

Damodaram, C., Giacomoni, M. H., Prakash Khedun, C., Holmes, H., Ryan, A., Saour, W., & Zechman, E. M., 2010. Simulation of Combined Best Management Practices and Low Impact Development for Sustainable Stormwater Management. JAWRA Journal of the American Water Resources Association, 46(5), 907-918.

Donofrio, J., Kuhn, Y., McWalter, K., & Winsor, M., 2009. Water-sensitive urban design: an emerging model in sustainable design and comprehensive water-cycle management. Environmental Practice, 11(03), 179-189.

FHWA, 2001. Federal Highway Administration, Introduction to Highway Hydraulics, Washington, D.C.

Flakne, R., Keller, R., 2012. Environmental Reviews and Case Studies: A Plan for Nature in Glenview: Creating and Implementing a Natural Resources Plan at the Community Level. Environmental Practice, 14, pp 35-44. doi:10.1017/S1466046611000482.

Freni G, Mannina G, Viviani G., 2010. Urban storm-water quality management: centralized versus source control. J Water Resour Plann Manage;136:268–78.

Heaney, J.P., Wright, L., Sample, D., 2002. Costs of Urban Stormwater Control. National Risk Management Resarch Laboratory. Office of Research and Development, USEPA (Chapter 6) (EPA-600/R-02/021).

Heiberger, J.R., 2013. Infiltration and Potential Groundwater Recharge Performance of Stormwater Bioretention Designed for Semiarid Climates. Master's Thesis. University of Utah: U.S.

Holman-Dodds, J. K., Bradley, A. A., & Potter, K. W., 2003. Evaluation of hydrologic benefits of infiltration based urban stormwater management. JAWRA Journal of the American Water Resources Association, 39(1), 205-215.

Hood, M. J., Clausen, J. C., & Warner, G. S., 2007. Comparison of stormwater lag times for low impact and traditional residential development. JAWRA Journal of the American Water Resources Association, 43(4), 1036-1046.

Jia, H., Lu, Y., Yu, S. L., & Chen, Y., 2012. Planning of LID–BMPs for urban runoff control: the case of Beijing Olympic Village. Separation and Purification Technology, 84, 112-119.

Leopold, L. B., 1968. Hydrology for Urban Land Planning – A Guidebook on the Hydrologic Effects of Urban Land Use. Geological Survey Circular: 554.

Low Impact Development Center, 2007. Urban Design Tools: Rain Barrels and Cisterns. Accessed: http://www.lid-stormwater.net/raincist_cost.htm#1

Maidment, D.R., 1993. The Hydrology Handbook. McGraw-Hill, Inc., 1993, pp. 5.1 – 5.39.

Mapes, J., Wolch, J., 2011. 'Living green': the promise and pitfalls of new sustainable communities. Journal of Urban Design, 16(1), 105-126.

McCuen, R. et al., 1996. Hydrology, FHWA-SA-96-067, Federal Highway Administration, Washington, DC.

Montalto F., Behr C., Alfredo K., Wolf M., Arye M., Walsh M., 2007. Rapid assessment of the cost-effectiveness of low impact development for CSO control. Landscape Urban Planning 82:117–131.

NOAA, 1982. NOAA Technical Report NWS 34, Mean Monthly, Seasonal, and Annual Pan Evaporation for the United States, Washington, D.C.

Palhegyi, G.E., 2010. Designing storm-water controls to promote sustainable ecosystems: science and application. Journal of Environmental Engineering;15:504–11.

Poff, N. L., Allan, J. D., Bain,M. B., Karr, J. R., Prestegaard, K. L., Richter, B. D., et al., 1997. The natural flow regime: a paradigm for river conservation and restoration. Bioscience, 47, 769–784.

Prince George's County, 1999. Maryland Department of Environmental Resources Programs and Planning Division, Low-Impact Development Design Strategies: An Integrated Design Approach.

Thurston, H.W. (Ed.), 2012. Economic Incentives for Stormwater Control. Boca Raton, FL: CRC Press Taylor & Francis Group.

UDEQ, 2013. Jordan River Total Maximum Daily Load Water Quality Study – Phase 1. Utah Department of Environmental Quality, Division of Water Quality. Accessed Online: http://www.waterquality.utah.gov/TMDL/Docs/09Sep/JordanRiverTMDL_Final_20130905.pdf

USDA, 2012. Accessed website March 8, 2013 – http://soildatamart.nrcs.usda.gov/Metadata.aspx?Survey=UT612&UseState=UT

USEPA, 1999. Preliminary Data Summary of Urban Stormwater Best Management Practices, EPA-821-R-99-012. Washington, DC. Accessed: http://www.epa.gov/OST/stormwater

USEPA, 2000. Low Impact Development (LID): A Literature Review. EPA-841-B-00-005, Office of Water, Washington, DC.

USEPA, 2007. Reducing Stormwater Costs through Low Impact Development (LID) Strategies and Practices. EPA 841-F-07-006. Accessed at http://water.epa.gov/polwaste/green/upload/2008_01_02_NPS_lid_costs07uments_reducingstormwatercosts-2.pdf

USEPA, 2012. Stormwater Management Model (SWMM): Version 5.0.022 with Low Impact Development (LID) Controls. Accessed: http://www.epa.gov/nrmrl/wswrd/wq/models/ swmm/

Utah, 2001. Utah's Water Resources: Planning for the Future. State of Utah, Natural Resources Division of Water Resources. Accessed at: www.nr.state.ut.us/wtrresc/waterplan/

Water Environment Research Foundation (WERF), 2009. BMP and LID Whole Life Cost Models: Version 2.0. Accessed at: http://www.werf.org/i/a/Ka/Search/ResearchProfile.aspx? ReportId=SW2R08

Watershed Management Institute (WMI), 1997. Operation, Maintenance, and Management of Stormwater Management Systems. Prepared for: US EPA Office of Water. Washington, DC.

Williams, E. S., Wise, W. R., 2006. Hydrologic impacts of alternative approaches to stormwater management and land development. JAWRA Journal of the American Water Resources Association, 42(2), 443-455.

Young, K.D., Dymond, R.L., and Kibler, D.F., 2011. Development of an improved approach for selecting storm-water best management practices. Journal of Water Resources Planning and Management, 137, 268-275.